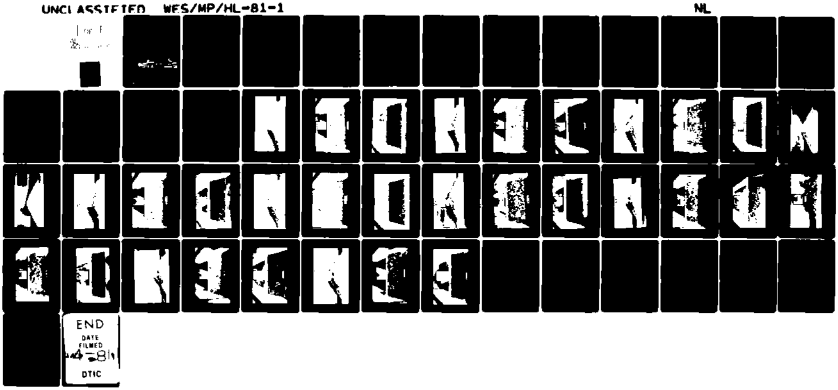
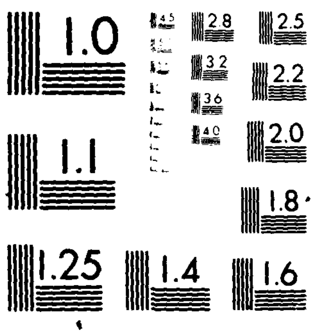
AD-A096 822 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG--ETC F/G 13/2
STABILITY OF RUBBLE-MOUND BREAKWATER, MAALAEA HARBOR, MAUI, HAW--ETC(U)
JAN 81 R D CARVER, D G MARKLE
UNCLASSIFIED WES/MP/HL-81-1 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

MISCELLANEOUS PAPER HL-81-1

STABILITY OF RUBBLE-MOUND BREAKWATER MAALAEA HARBOR, MAUI, HAWAII

Hydraulic Model Investigation

by

Robert D. Carver, Dennis G. Markle

Hydraulics Laboratory
U. S. Army Engineer Waterways Experiment Station
P. O. Box 631, Vicksburg, Miss. 39180

January 1981

Final Report

Prepared for U. S. Army Engineer Division, Pacific Ocean
Fort Shafter, Hawaii 96858

14 WES/MP/HL-81-1

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER
Miscellaneous Paper HL-81-1

2. GOVT ACCESSION NO.
AD-A096822

3. RECIPIENT'S CATALOG NUMBER
9

4. TITLE (and Subtitle)
6 STABILITY OF RUBBLE-MOUND BREAKWATER, MAALAEA HARBOR, MAUI, HAWAII; Hydraulic Model Investigation.

5. TYPE OF REPORT & PERIOD COVERED
Final report. Apr-Jun 80

6. PERFORMING ORG. REPORT NUMBER

7. AUTHOR(s)
10 Robert D. Carver
Dennis G. Markle

8. CONTRACT OR GRANT NUMBER(s)

9. PERFORMING ORGANIZATION NAME AND ADDRESS
U. S. Army Engineer Waterways Experiment Station
Hydraulics Laboratory
P. O. Box 631, Vicksburg, Miss. 39180

10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS

11. CONTROLLING OFFICE NAME AND ADDRESS
U. S. Army Engineer Division, South Pacific
Building 230
Fort Shafter, Hawaii 96858

12. REPORT DATE
January 1981

13. NUMBER OF PAGES
50

14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)
12 55

15. SECURITY CLASS. (of this report)
Unclassified

15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)
Breakers (Water waves)
Breakwaters
Hydraulic models
Maalaea Harbor, Hawaii
Rubble-mound breakwaters

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

An undistorted-scale hydraulic model study was conducted to investigate the adequacy of three breakwater cross sections considered for use at Maalaea Harbor, Maui, Hawaii. Plan 1 was protected by 6-ton dolosse on both the sea side and beach side, whereas Plans 2 and 3 used 6-ton dolosse sea side and 7- to 10-ton stone beach side. Plans 1 and 2 used 1V- on 2H-armor slopes and Plan 3 used 1V- on 1.5H-armor slopes. Based on results of model tests, it was concluded that

(Continued)

DD FORM 1473, 1 JAN 73 — EDITION OF 1 NOV 65 IS OBSOLETE

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

411389

20. ABSTRACT (Continued).

Plans 1, 2, and 3 are stable designs for the maximum breaking wave heights that can be expected to occur for 12- to 16-sec waves at swl's of -1 and +4 ft mllw. Also, Plans 2 and 3 can withstand attack of 16-sec, 15.2-ft breaking waves at an swl of +6 ft mllw and 16-sec, 16.7-ft breaking waves at an swl of +8 ft mllw without experiencing significant damage.

Preface

The model investigation reported herein was requested by the U. S. Army Engineer Division, Pacific Ocean (POD), in a letter to the U. S. Army Engineer Waterways Experiment Station (WES) dated 7 February 1980. The investigation was authorized by POD Intra-Army Order PODSP-CIV-80-21 dated 7 March 1980.

Model tests were conducted at WES during the period April through June 1980, under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and Dr. R. W. Whalin, Chief of the Wave Dynamics Division. Tests were conducted by Messrs. R. D. Carver and D. G. Markle, Research Hydraulic Engineers, and Mr. C. R. Herrington, Engineering Technician. Execution of this study and preparation of this report were performed by Messrs. Carver and Markle under the supervision of Mr. D. D. Davidson, Chief of the Wave Research Branch.

Liaison between POD and WES was maintained during the course of the investigation by telephone conversations and progress reports.

Commander and Director of WES during the conduct of the study and the preparation and publication of this report was COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

Contents

	Page
Preface	1
Conversion Factors, U. S. Customary to Metric (SI) Units of Measurement	3
Introduction	4
Purpose of Model Study	4
Design of Model	4
Test Equipment	6
Method of Constructing Test Sections	6
Description of Plans 1 and 2	7
Selection of Test Conditions	7
Test Results for Design Wave Conditions	8
Safety-Factor Tests of Plan 2	9
Rationale and Test Results for Plan 3	10
Conclusions	12
Photos 1-32	
Plates 1-6	

Conversion Factors, U. S. Customary to Metric (SI) Units of Measurement

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2000 lb, mass)	907.1847	kilograms

STABILITY OF RUBBLE-MOUND BREAKWATER

MAALAEA HARBOR, MAUI, HAWAII

Hydraulic Model Investigation

Introduction

1. Maalaea Harbor is located on the west coast of Maui, one of the larger islands in the Hawaiian Island group (Plate 1). Pleasure boating, sport fishing, and commercial fishing have increased significantly during recent years and expansion of an existing small-boat harbor has been proposed. In order to provide wind wave and swell protection for the expanded harbor, it will be necessary to extend the existing south breakwater.

Purpose of Model Study

2. The original purpose of the model study was to experimentally investigate the adequacy of two breakwater sections proposed by the U. S. Army Engineer Division, Pacific Ocean (POD), for the south breakwater extension. Both sections used 1V- on 2H-armor slopes. One alternative was protected by 6-ton* dolosse on both the sea side and beach side, whereas the other employed 6-ton dolosse sea side and 7- to 10-ton stone beach side. Later, following completion of tests for the two original plans, it was decided to also investigate a third plan using 1V- on 1.5H-armor slopes in an attempt to reduce costs for construction of the breakwater.

Design of Model

3. Tests were conducted at an undistorted linear scale of 1:27.5, model to prototype. Scale selection was based on the size of

---

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

model armor units available relative to the estimated size of prototype armor units required for stability, the elimination of stability scale effects* and capabilities of the available wave tank. Based on Froude's model law** and the linear scale of 1:27.5, the following model-to-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

Characteristics	Dimensions	Model:Prototype Scale Relations
Length	L	$L_r = 1:27.5$
Area	L^2	$A_r = L_r^2 = 1:756$
Volume	L^3	$V_r = L_r^3 = 1:20{,}797$
Time	T	$T_r = L_r^{1/2} = 1:5.24$

4. The specific weight of water used in the model was assumed to be 62.4 pcf and that of seawater is 64.0 pcf. Also, specific weights of model breakwater construction materials were not the same as their prototype counterparts. These variables were related using the following transference equation:

$$\frac{(W_r)_m}{(W_r)_p} = \frac{(\gamma_r)_m}{(\gamma_r)_p} \left(\frac{L_m}{L_p}\right)^3 \left[\frac{(S_r)_p - 1}{(S_r)_m - 1}\right]^3$$

where

Subscripts m, p = model and prototype quantities, respectively

W_r = weight of an individual armor unit or stone, lb

γ_r = specific weight of an individual armor unit or stone, pcf

---

* R. Y. Hudson. 1975 (Jun). "Reliability of Rubble-Mound Breakwater Stability Models; Hydraulic Model Investigation," Miscellaneous Paper H-75-5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

** J. C. Stevens et al. 1942. "Hydraulic Models," Manuals on Engineering Practice No. 25, American Society of Civil Engineers, New York, N. Y.

L_m/L_p = linear scale of the model

S_r = specific gravity of an individual armor unit or stone relative to the water in which the breakwater is constructed, i.e., $S_r = \gamma_r/\gamma_w$

γ_w = the specific weight of water, pcf

5. Crown protection for all plans will be provided by a cast-in-place concrete rib cage. It was assumed that the prototype rib cage will be stable; therefore, it was not necessary that the model rib cage be dynamically similar to the prototype. The model rib cage, constructed of Plexiglas, was geometrically similar to the prototype, thus ensuring proper reflection and dissipation of incident wave energy.

Test Equipment

6. Tests were conducted in a portion (100 ft long, 5 ft wide, and 3 ft deep) of an L-shaped concrete flume, which has overall dimensions of 250 ft long, 50 and 80 ft wide at the top and bottom of the L, respectively, and 4.5 ft deep. The flume layout is shown in Plate 2. A 44-ft (model) length of 1V-on-50H slope, representative of the existing prototype sea bottom, was molded and test sections were installed 29 ft beachward of the slope's toe. The test facility is equipped with a flap-type wave generator, capable of producing sinusoidal waves of various periods and heights. Test waves of the required characteristics were generated by varying the frequency and amplitude of the plunger motion. Changes in water-surface elevation as a function of time were measured by electrical wave-height gages in the vicinity of where the toe of the test sections was to be placed and recorded on chart paper by an electrically operated oscillograph. Measurement of wave heights (generator calibration) without test sections in place simulated existing conditions.

Method of Constructing Test Sections

7. Model breakwater sections were constructed to reproduce, as closely as possible, results of the usual methods of constructing

prototype structures. Core material, dampened as it was dumped by bucket or shovel into the flume, was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype breakwater. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. Underlayer stone was then added by shovel and smoothed to grade by hand or with trowels but was not packed in place. Armor units used in the cover layer were placed in a random manner, i.e., laid down in such a way that no intentional interlocking of the units was obtained. Model elevations were controlled with an engineer's level to a tolerance of ±0.005 ft.

Description of Plans 1 and 2

8. Plan 1, shown in Plate 3 and Photos 1-3, was constructed to a crown elevation of +13.0 ft mean lower low water (mllw) and used armor slopes of 1V on 2H both sea side and beach side. Crown protection was provided by a concrete rib cage, underlain with one layer of 6- to 8-ton stone. Individual concrete ribs were 15 ft long, 6 ft high, and 3 ft wide connected by two 3.0- × 1.5- × 1.5-ft concrete spacers which were flush with the top of the ribs at one-third points along the 15-ft length. The ribs used a 6-ft spacing center-to-center and in the prototype the bottom of each rib will be doweled to the stone underlayer. Both seaward and beachward slope protection was provided by 6-ton dolosse. Core stone weights of 100 to 500 lb were used, and the armor and core were interfaced with 1- to 2-ton underlayer stone. Plan 2 (Plate 4 and Photos 4-6) was the same as Plan 1 except that 7- to 10-ton stone was used to armor the beach side of the breakwater.

Selection of Test Conditions

9. Based on anticipated prototype wave conditions, it was decided that stability tests should consider 12-, 14-, and 16-sec wave periods at still-water levels (swl's) of -1.0 and +4.0 ft mllw. Model

observations indicated that for the selected wave periods and swl's, the corresponding maximum breaking wave was always more severe than any lesser wave height. Observations of incident wave forms at the structure showed that the worst breaking waves (as a function of wave period) which could be made experimentally to attack the section for the selected conditions were as follows:

swl ft mllw	Wave Period sec	Worst Breaking Wave Height ft
-1.0	12.0	8.9
-1.0	14.0	9.2
-1.0	16.0	10.5
+4.0	12.0	12.5
+4.0	14.0	12.6
+4.0	16.0	13.5

Model observations also indicated that due to consistently larger breaking wave heights, the 16-sec period condition was more severe than either the 12- or 14-sec period conditions. Wave heights at the 12- and 14-sec periods appeared to be very similar in severity. It was decided that for the range of wave conditions considered, the stability response of the test sections could be adequately evaluated by subjecting the test structures to the following storm surge hydrograph:

Step	swl ft mllw	Test Wave Period, sec	Test Wave Height, ft	Prototype Duration, hr	Wave Type
	-1.0	14.0	4.6	0.33	Shakedown
1	-1.0	14.0	9.2	1.00	Worst breaking
2	-1.0	16.0	10.5	1.00	Worst breaking
3	+4.0	14.0	12.6	1.00	Worst breaking
4	+4.0	16.0	13.5	1.00	Worst breaking
5	-1.0	14.0	9.2	1.00	Worst breaking
6	-1.0	16.0	10.5	1.00	Worst breaking

The above hydrograph is graphically depicted in Plate 5.

Test Results for Design Wave Conditions

10. During testing of Plan 1 some intermittent minor rocking of

a few sea-side armor units was observed throughout the hydrograph and moderate wave overtopping during steps 3 and 4 caused minor rocking of a few beach-side armor units. Photos 7-9 show the test section after wave attack. A comparison of before- and after-test photographs shows that the final stabilized condition of the structure is virtually indistinguishable from its original appearance.

11. Plan 2 also exhibited an excellent stability response. Minor rocking of a few sea-side armor units was observed throughout the hydrograph; however, no displaced damage occurred. Even though moderate wave overtopping was present during steps 3 and 4, no damage was incurred by the 7- to 10-ton, beach-side armor stone. Photo 10 shows a 16-sec, 13.5-ft wave (step 4) impinging on the breakwater and Photo 11 shows the wave overtopping the structure. Photos 12-14 show the structure after completion of the hydrograph. Test results of Plans 1 and 2 were verified by a complete reconstruction and retesting.

Safety-Factor Tests of Plan 2

12. In designing rubble-mound breakwaters, or with any engineered structures, it is advantageous to determine what margin of safety is present in the selected designs. Consequently, at the conclusion of the repeat hydrograph test of Plan 2, it was decided to subject the structure to storm surges and wave heights in excess of the maximum design condition (16-sec, 13.5-ft waves at an swl of +4 ft mllw). A check of calibration data revealed that for the maximum design wave period of 16 sec, the wave generator was capable of producing depth-limited breaking waves for swl's up to +8 ft mllw. Therefore, even though POD realized swl's above the +4 ft mllw were extreme events, they requested Plan 2 be tested with 16-sec breaking waves at swl's of +6 and +8 ft mllw. Observations of incident wave forms at the structure showed that the worst breaking wave conditions which could be made experimentally to attack the section at the +6 and +8 ft swl's had heights of 15.2 and 16.7 ft, respectively.

13. Following completion of the repeat hydrograph test of Plan 2,

the water level was raised to +6 ft mllw and the structure was subjected to 16-sec, 15.2-ft waves. This condition produced significant wave overtopping, resulting in displacement of two beach-side armor units. In-place rocking of 1 to 2 percent of the sea-side armor units (total number of dolosse used on the sea side of the model cross section was approximately 300 units) was observed; however, no displaced damage was experienced. Photos 15-17 show the after-testing condition of the breakwater.

14. Upon completion of testing at the +6 ft swl, the water level was raised to +8 ft mllw and the test section was subjected to attack of 16-sec, 16.7-ft waves. This condition produced major wave overtopping. Water coming across the structure did not directly strike exposed armor, and no further dislocation of beach-side armor was observed. Several sea-side armor units between +8 and +13 ft mllw shifted in their original positions, and 2 to 3 percent of the dolosse were observed to rock in place; however, no displaced damage resulted. Photos 18-20 show the final stability condition of the structure.

15. Plan 2 was subjected to wave attack for 2 hr (prototype) at each swl investigated. This duration of wave attack allowed sufficient time for the structure to stabilize, i.e., time for all movement of armor to abate.

Rationale and Test Results for Plan 3

16. Safety-factor tests of Plan 2 demonstrated that the structure was able to withstand attack of waves well in excess of the maximum design wave height; therefore, it was decided to investigate alternative schemes that might substantially reduce the structure's cost without sacrificing its functional performance. Some of the factors that govern material volume and, therefore, initial construction costs are elevation and width of the crown, type and weight of armor, and slope on which the armor is placed. Based on discussions between POD and the U. S. Army Engineer Waterways Experiment Station (WES), it was decided that in this particular study the greatest volumetric reductions with the least

probable impact on functionality could probably be achieved by steepening both seaward and beachward armor slopes from 1V on 2H to 1V on 1.5H. Based on this rationale, Plan 3 (Plate 6 and Photos 21-23) was identical with Plan 2 except for the steeper slopes.

17. Stability test results for Plan 3 were favorable. Minor rocking of a few sea-side armor units was observed throughout the hydrograph; however, no displaced damage occurred. Beach-side armor was undamaged, even though substantial wave overtopping was present during steps 3 and 4. It should be noted that as opposed to Plans 1 and 2, there appeared to be an increase in wave overtopping during attack of 14-sec, 12.6-ft waves (step 3) and 16-sec, 13.5-ft waves (step 4). Photos 24-26 show the breakwater after completion of the hydrograph. Reconstruction and retesting of the model section verified results of the initial test.

18. Plan 3 also was tested with 16-sec, 15.2-ft breaking waves at an swl of +6 ft mllw and 16-sec, 16.7-ft breaking waves at an swl of +8 ft mllw. Attack of the 16-sec, 15.2-ft breaking waves produced in-place rocking of 2 to 3 percent of the sea-side armor units; however, no displaced damage was observed. Beach-side armor was stable, even though major wave overtopping was experienced. Photos 27-29 show the after-testing condition of the structure. Wave attack at the +8 ft swl caused several sea-side armor units above the swl to shift in their original positions and 3 to 4 percent of the dolosse rocked in place; however, no displaced damage occurred. Also, beach-side armor stone was again stable. Photos 30-32 show the final stability condition of the structure.

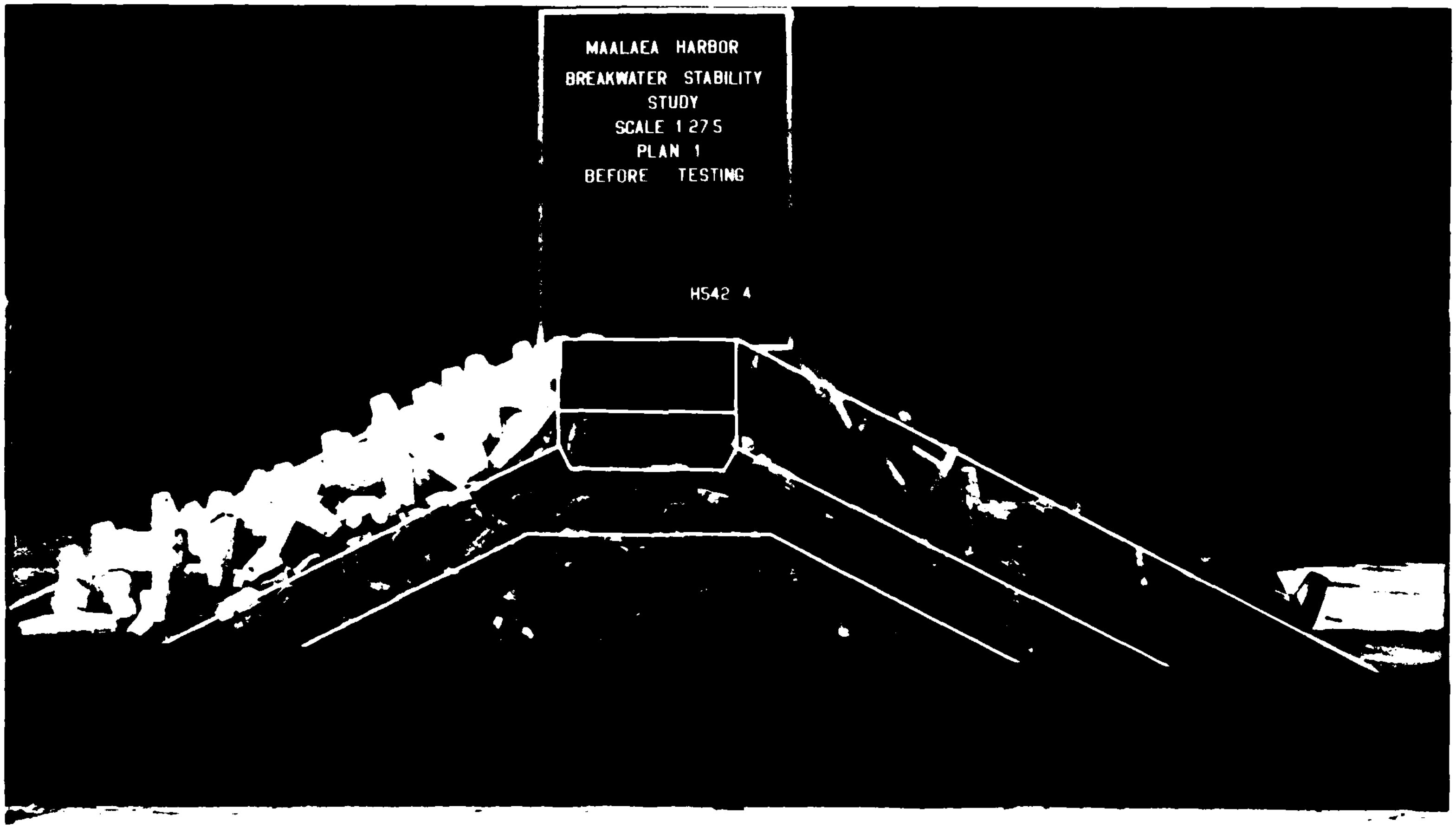
19. As with Plan 2, Plan 3 was subjected to wave attack for 2 hr (prototype) at each of the higher swl's (+6 ft mllw and +8 ft mllw). Again, this duration of wave attack allowed sufficient time for the structure to stabilize.

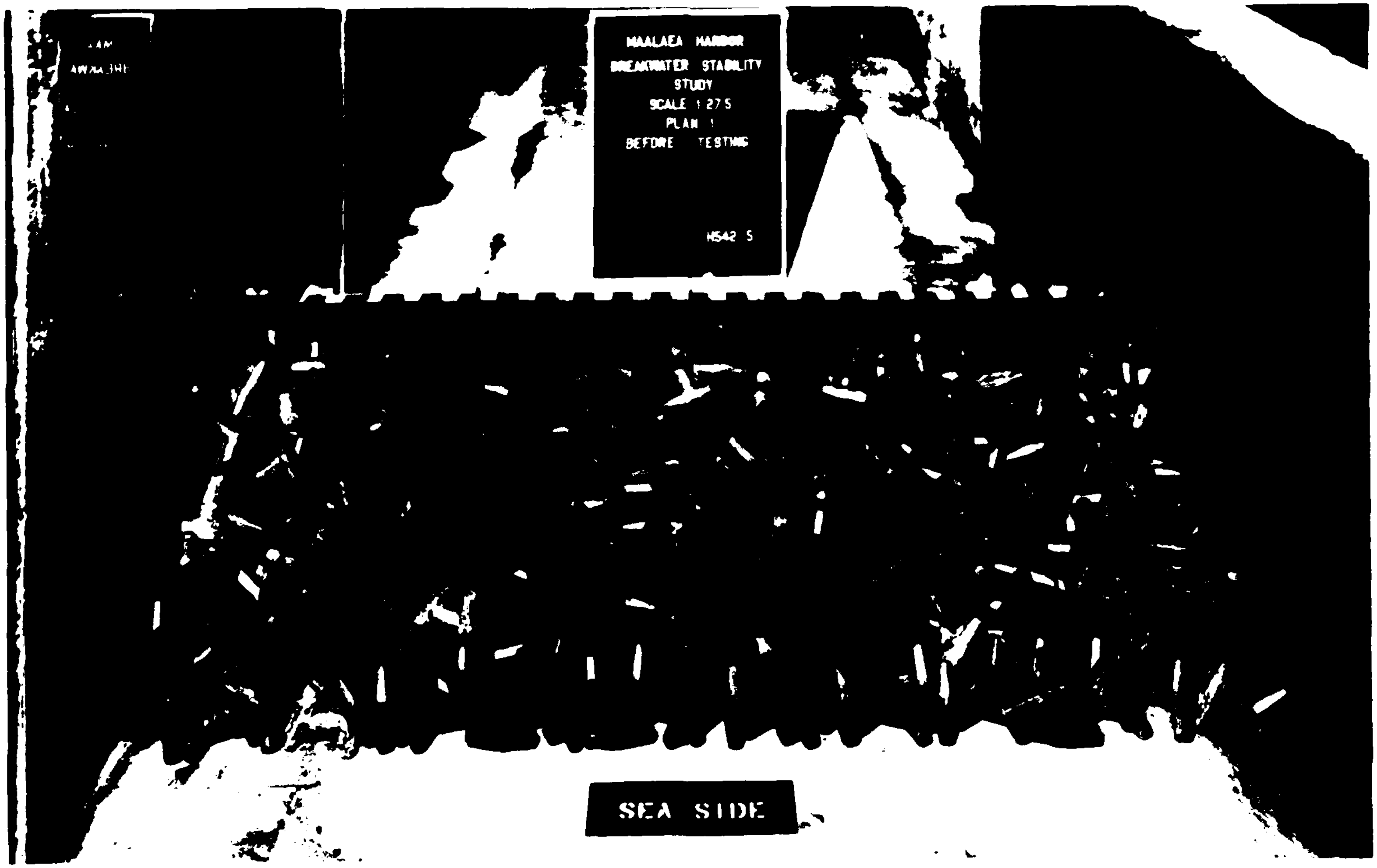
20. Stability test results of Plans 2 and 3 for the +6 and +8 ft swl's were very similar with slightly more in-place rocking of dolosse observed with Plan 3. Also, a moderate increase in wave overtopping and wave transmission was observed with Plan 3.

Conclusions

21. Based on the assumptions, tests, and results reported herein, it is concluded that:

- a. Plans 1, 2, and 3 are stable designs for the maximum breaking wave heights that can be expected to occur for 12- to 16-sec waves at swl's of -1 and +4 ft mllw.
- b. For safety-factor tests using extreme swl's, Plans 2 and 3 can withstand attack of 16-sec, 15.2-ft breaking waves at an swl of +6 ft mllw and 16-sec, 16.7-ft breaking waves at an swl of +8 ft mllw without experiencing significant damage.
- c. Stability test results of Plans 2 and 3 are very similar with slightly more in-place rocking of dolosse observed at the +6 and +8 ft swl's for Plan 3.
- d. As opposed to Plan 2, Plan 3 exhibited increased wave overtopping and wave transmission at the +4, +6, and +8 ft swl's.


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1 27 5
PLAN 1
BEFORE TESTING
H542 4


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 1
BEFORE TESTING
H542 5
SEA SIDE

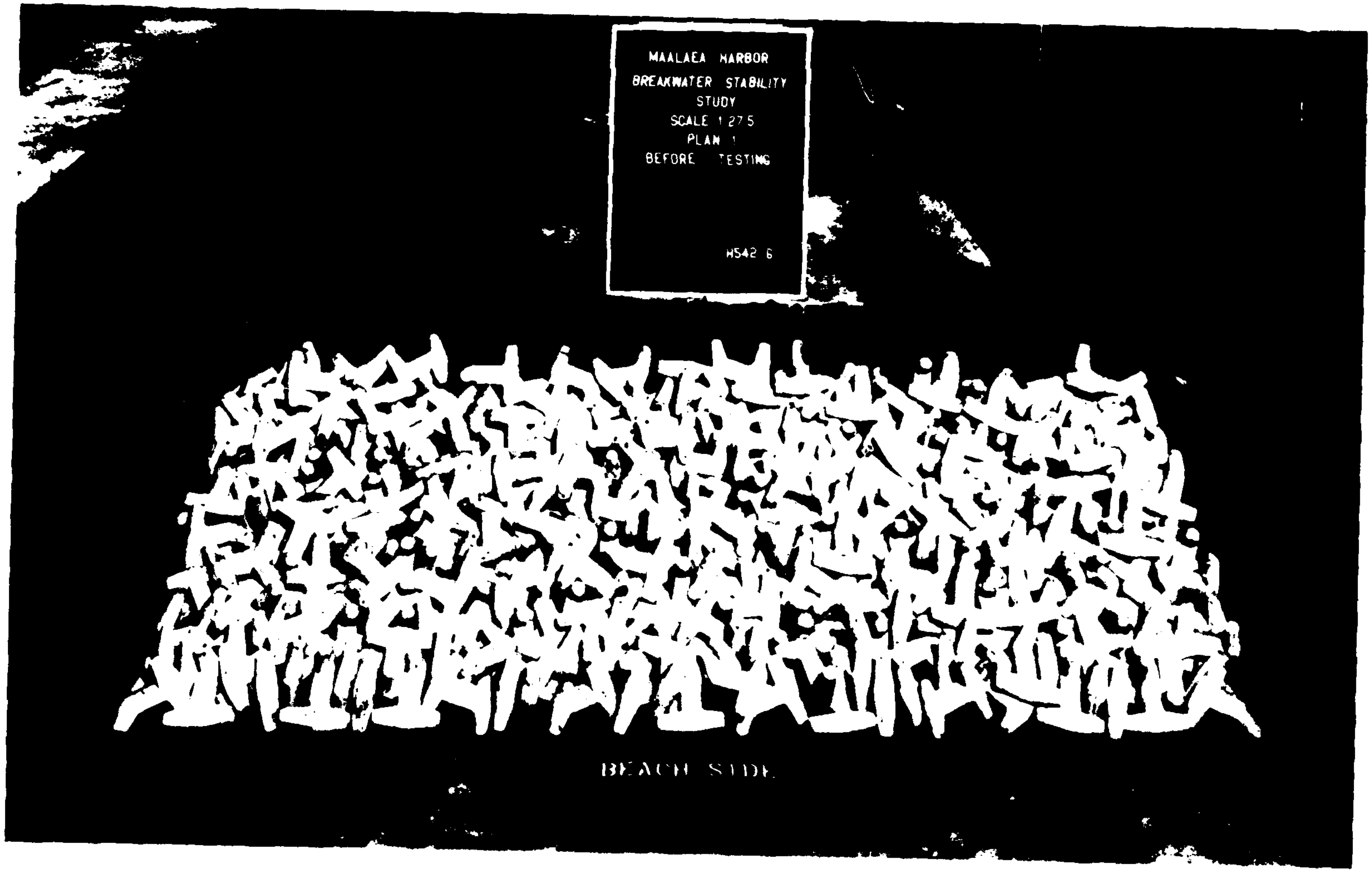


Photo 3. Beach-side view of Plan 1 before wave attack

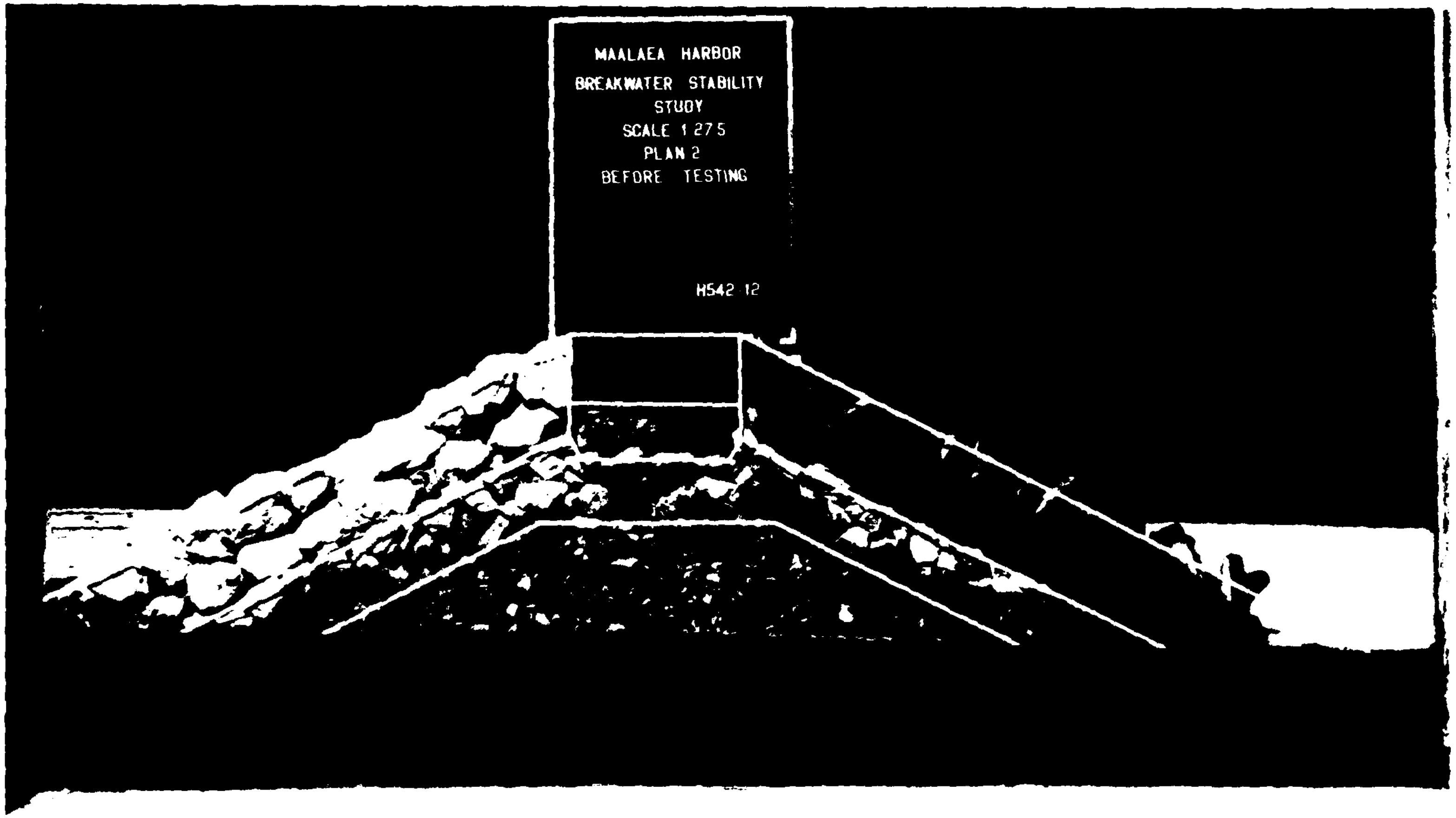
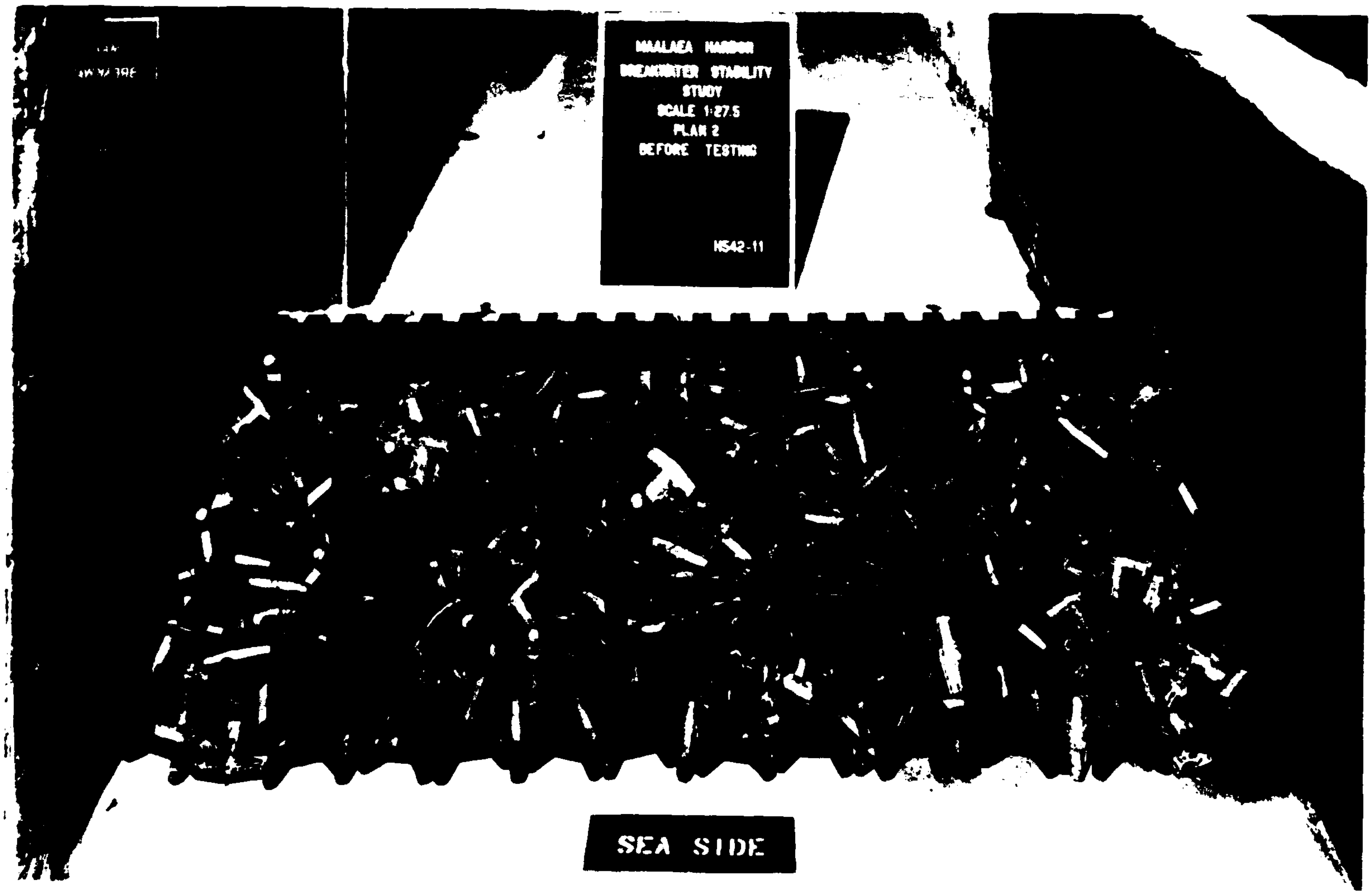


Photo [illegible]. Side view of Plan 2 before wave attack


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 2
BEFORE TESTING
H542-11
SEA SIDE

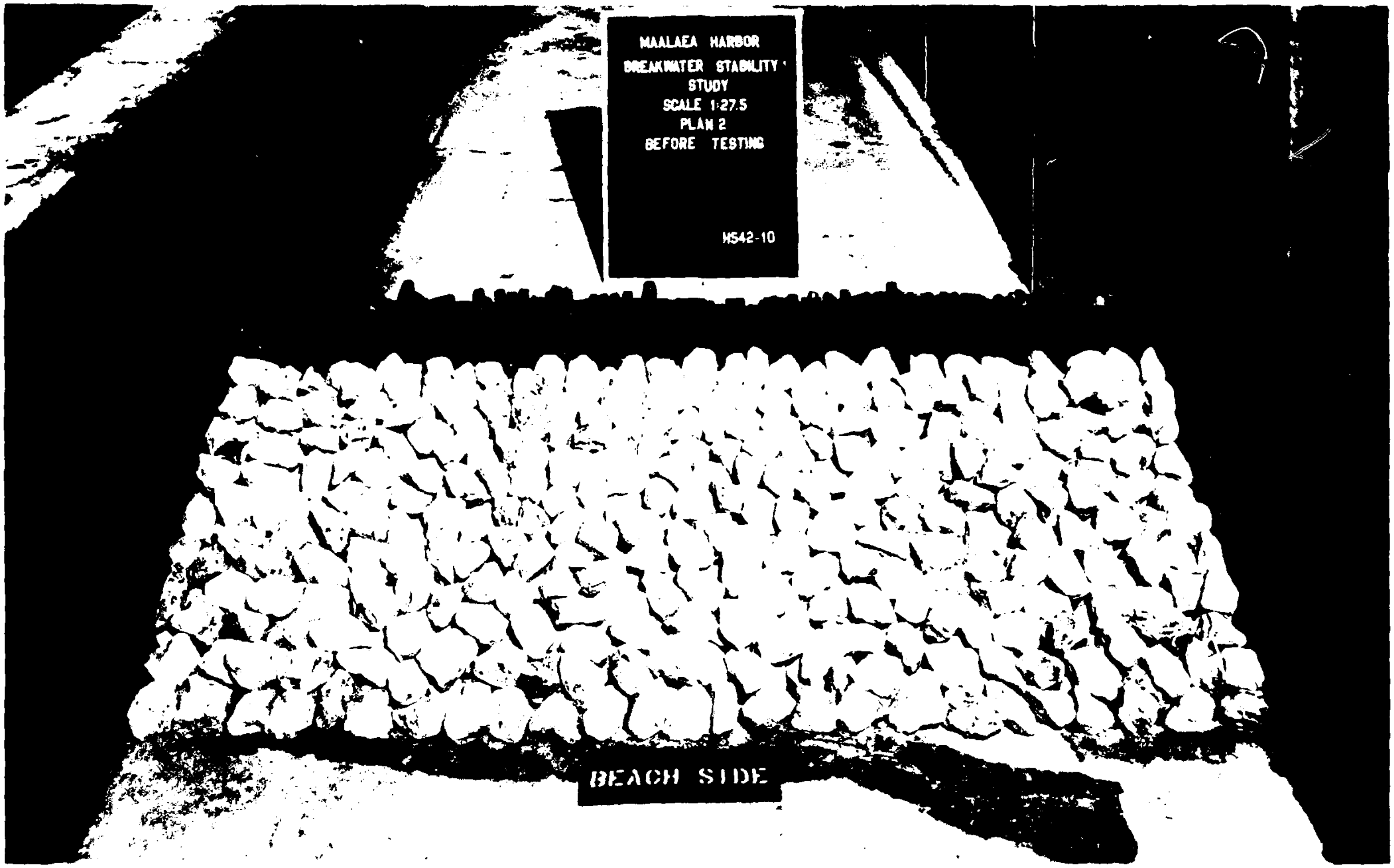
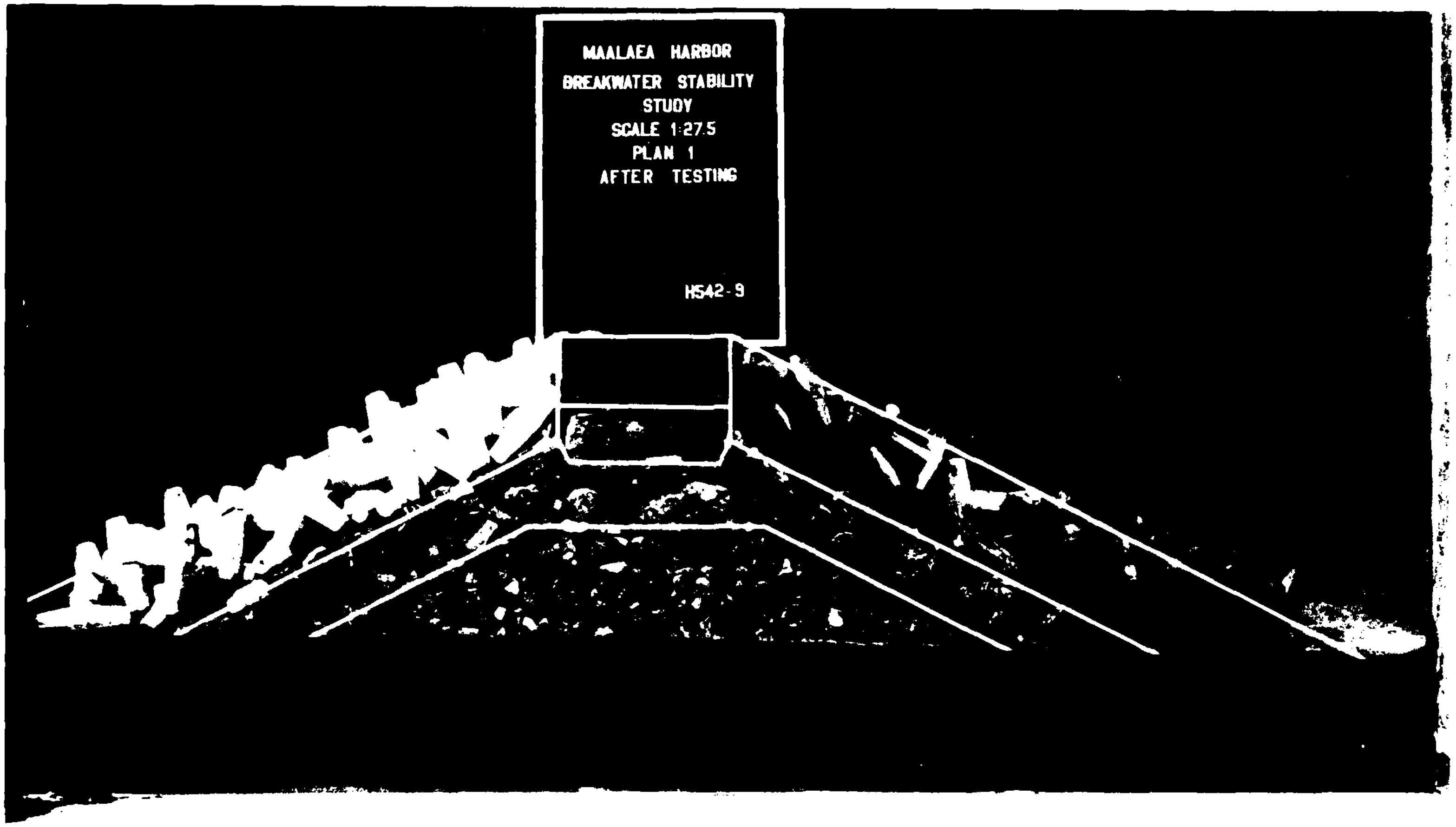


Photo 6. Beach-side view of Plan 2 before wave attack


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 1
AFTER TESTING
H542-9

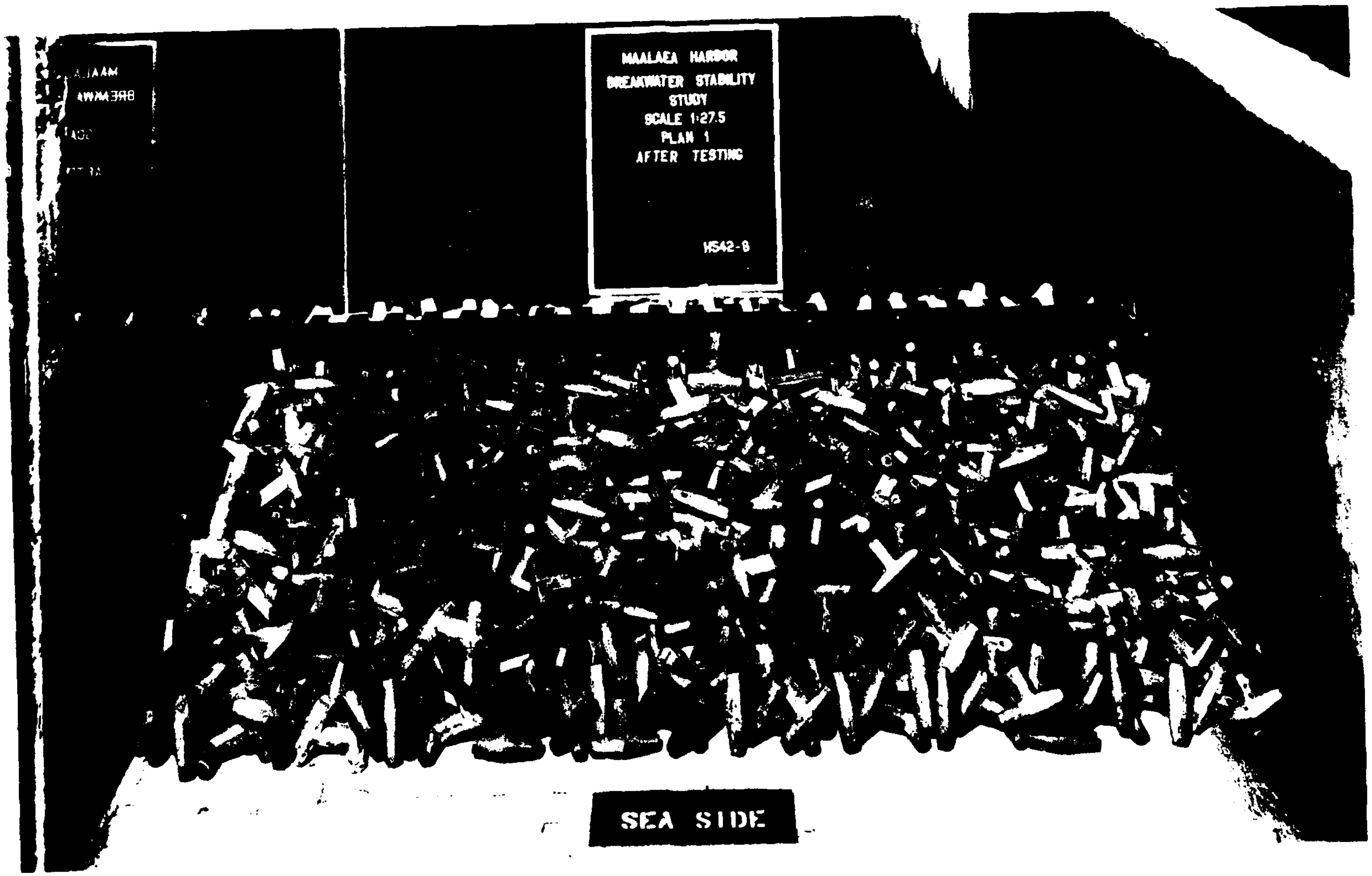
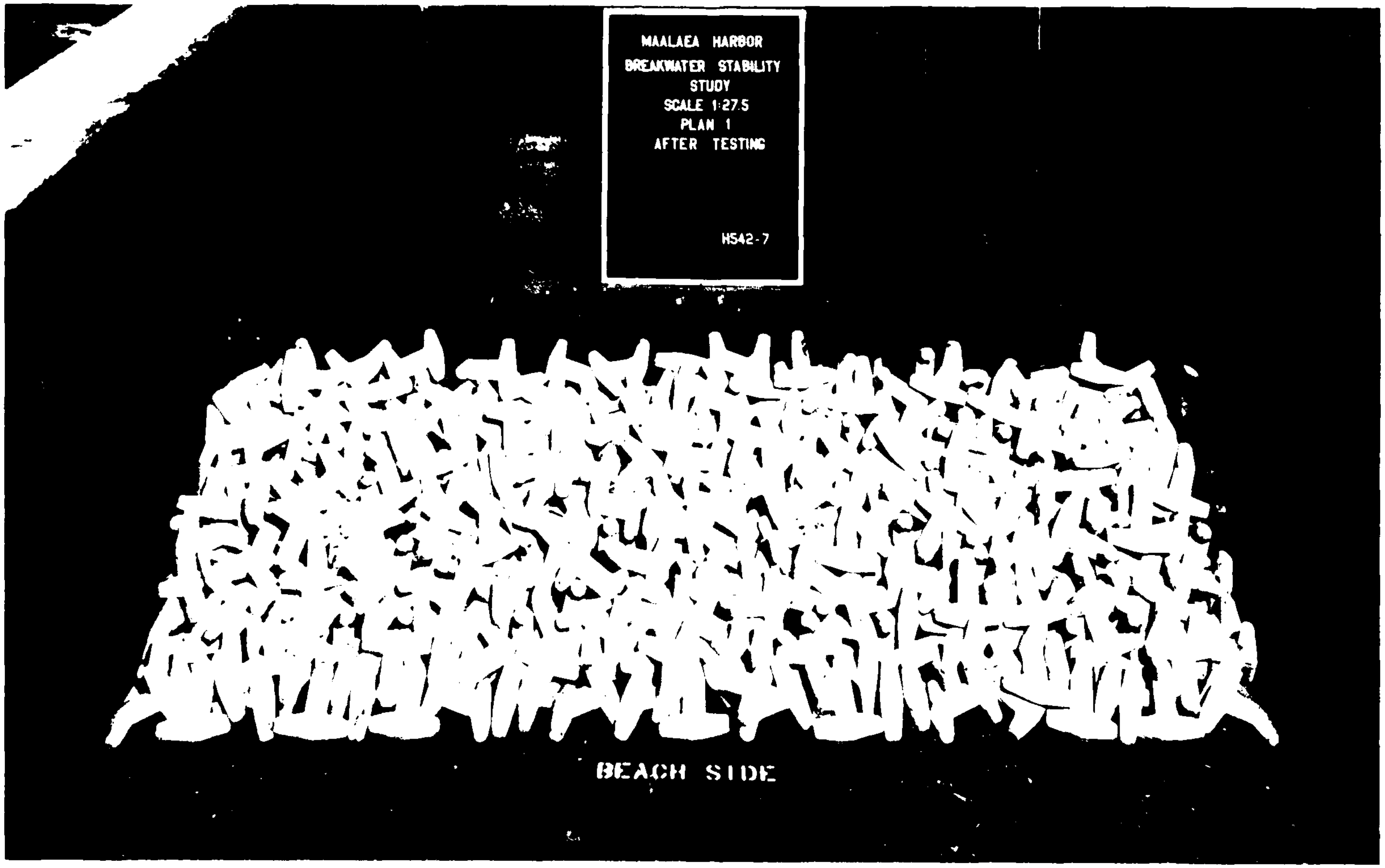
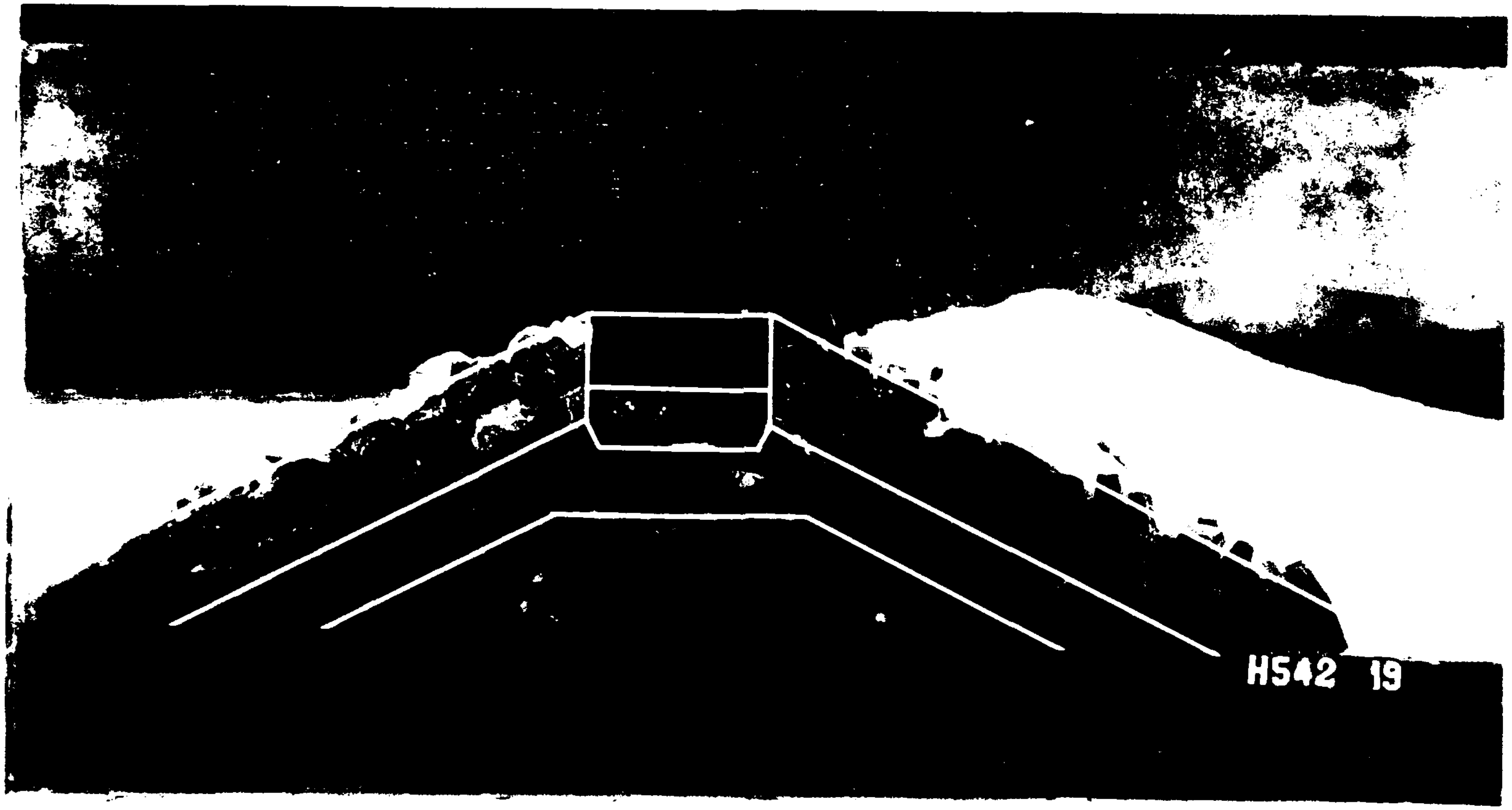


Photo [illegible]. Sea-side view of Plan 1 after completion of storm-wave hydrograph


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 1
AFTER TESTING
H542-7
BEACH SIDE


H542 19

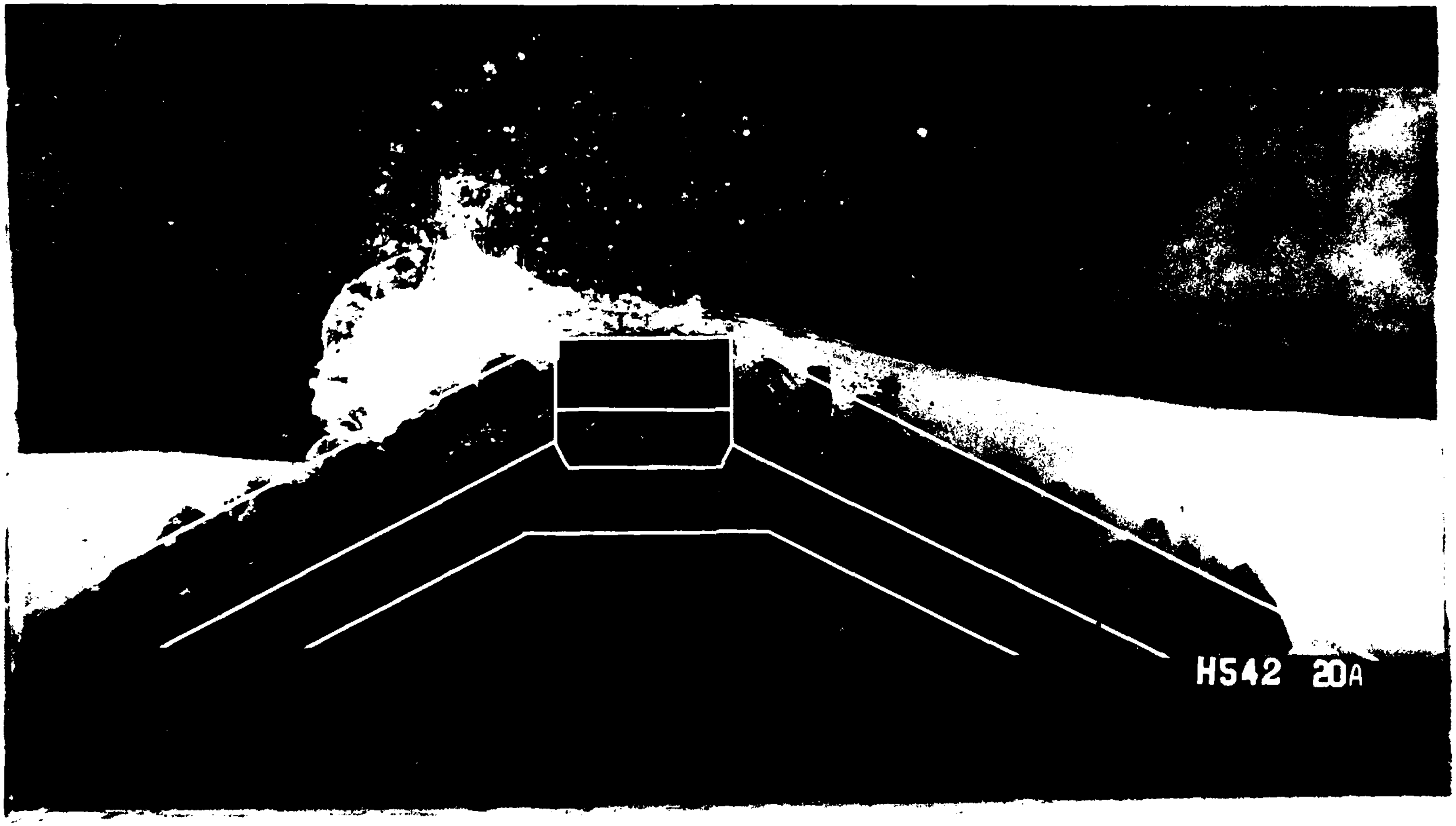
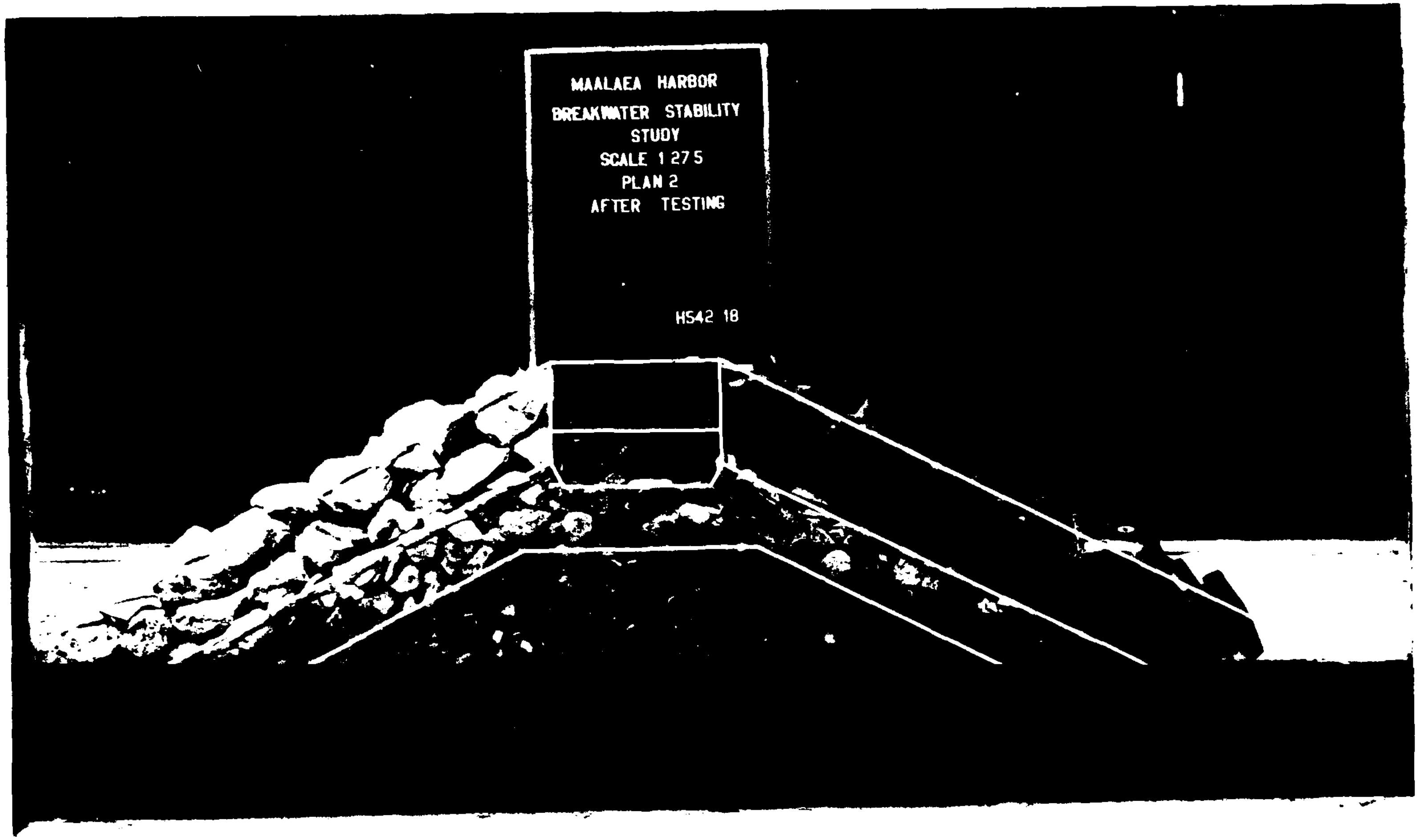


Photo 11. Side view of Ham. 2 being overtopped by a [illegible]



[illegible]. Side view of Plan 2 after completion of storm-wave hydrograph

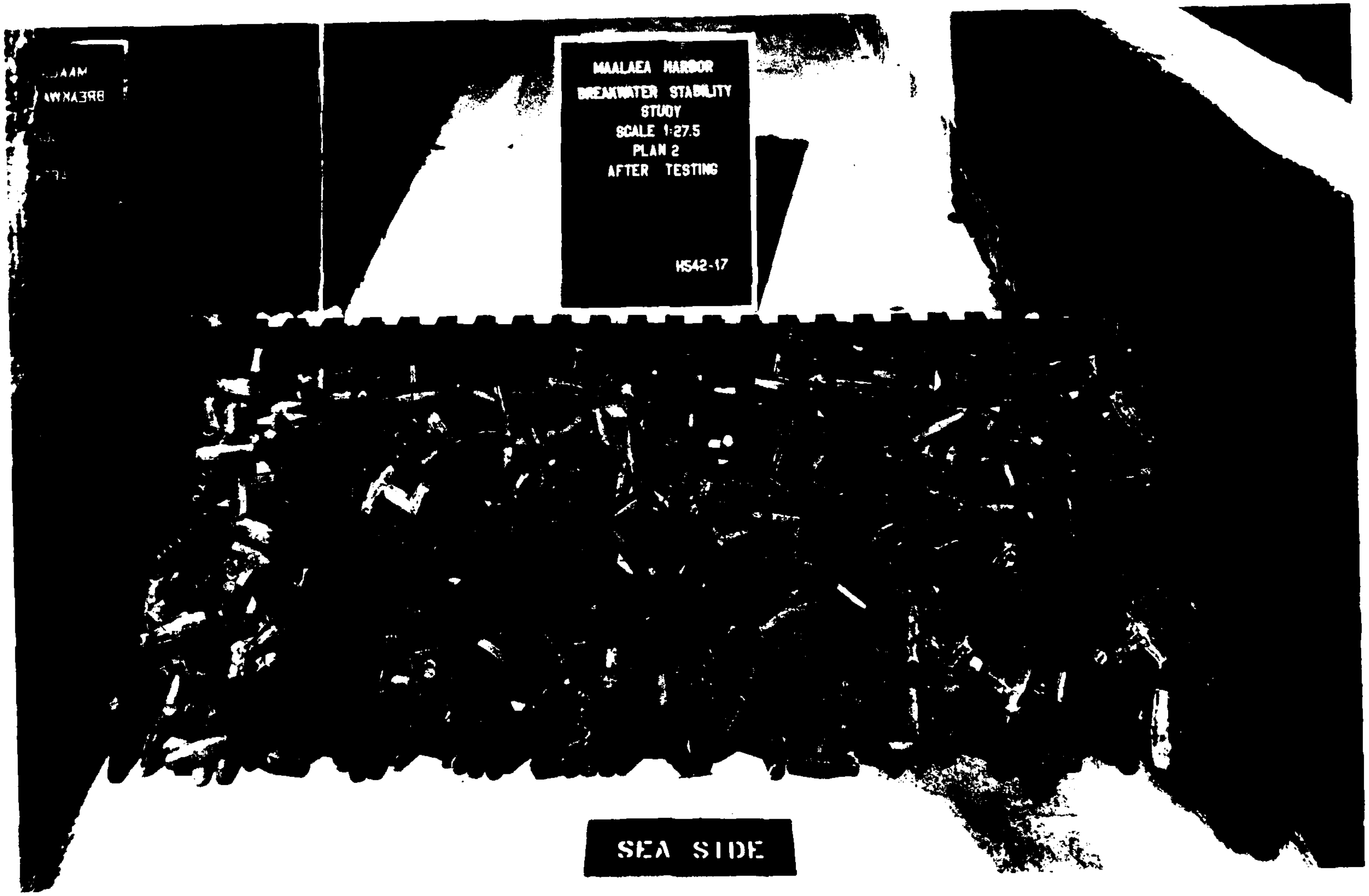
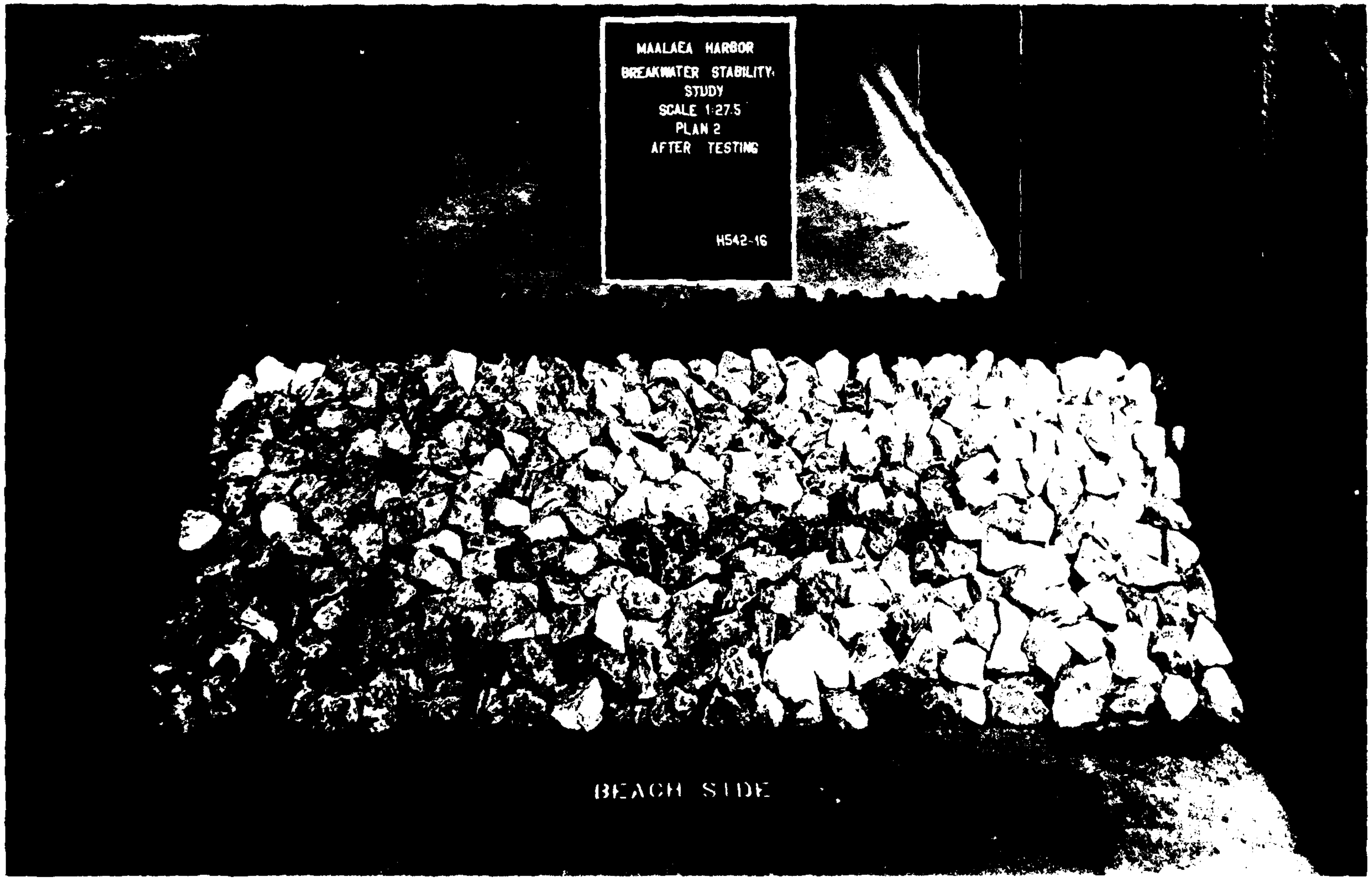
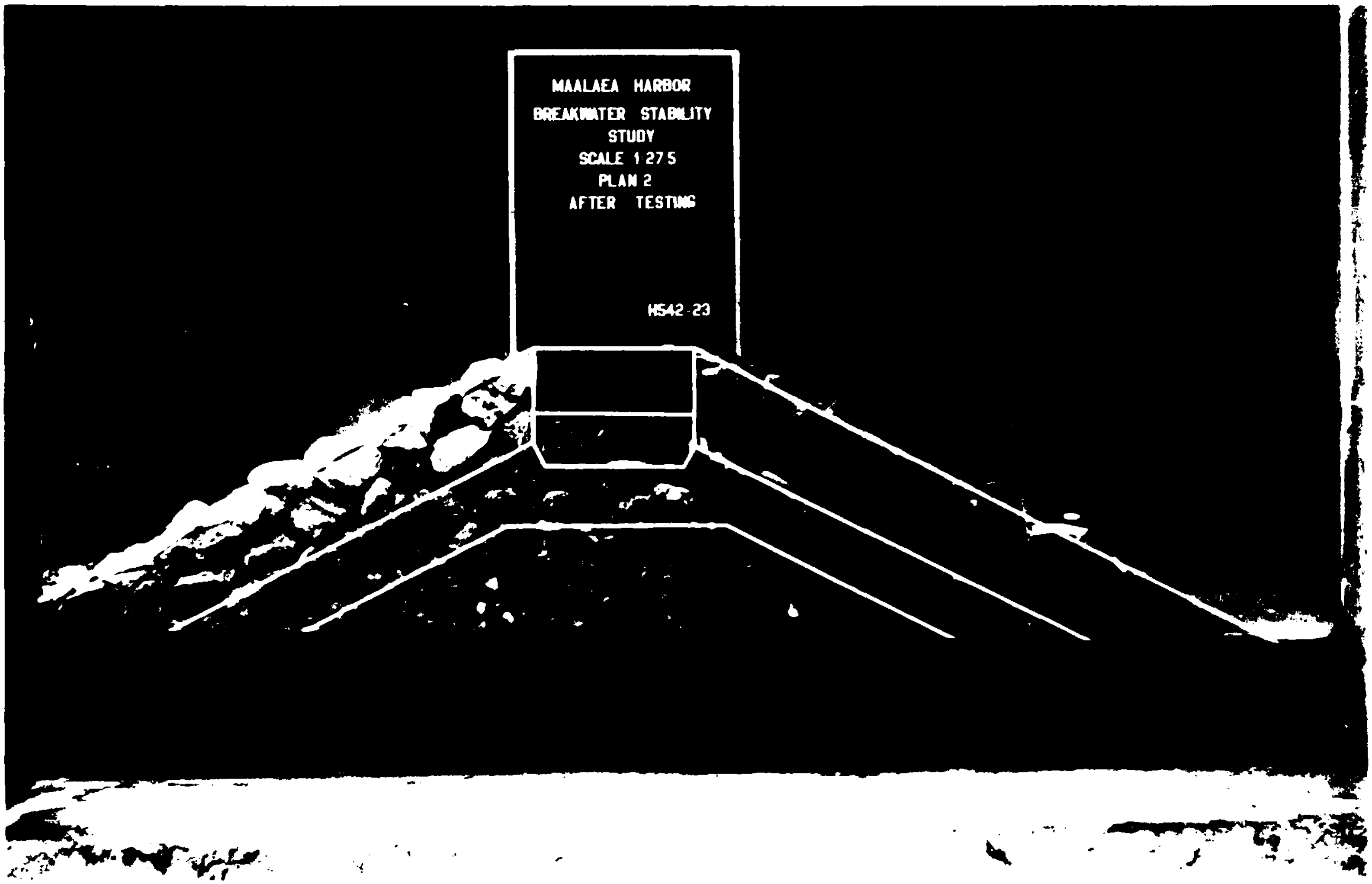
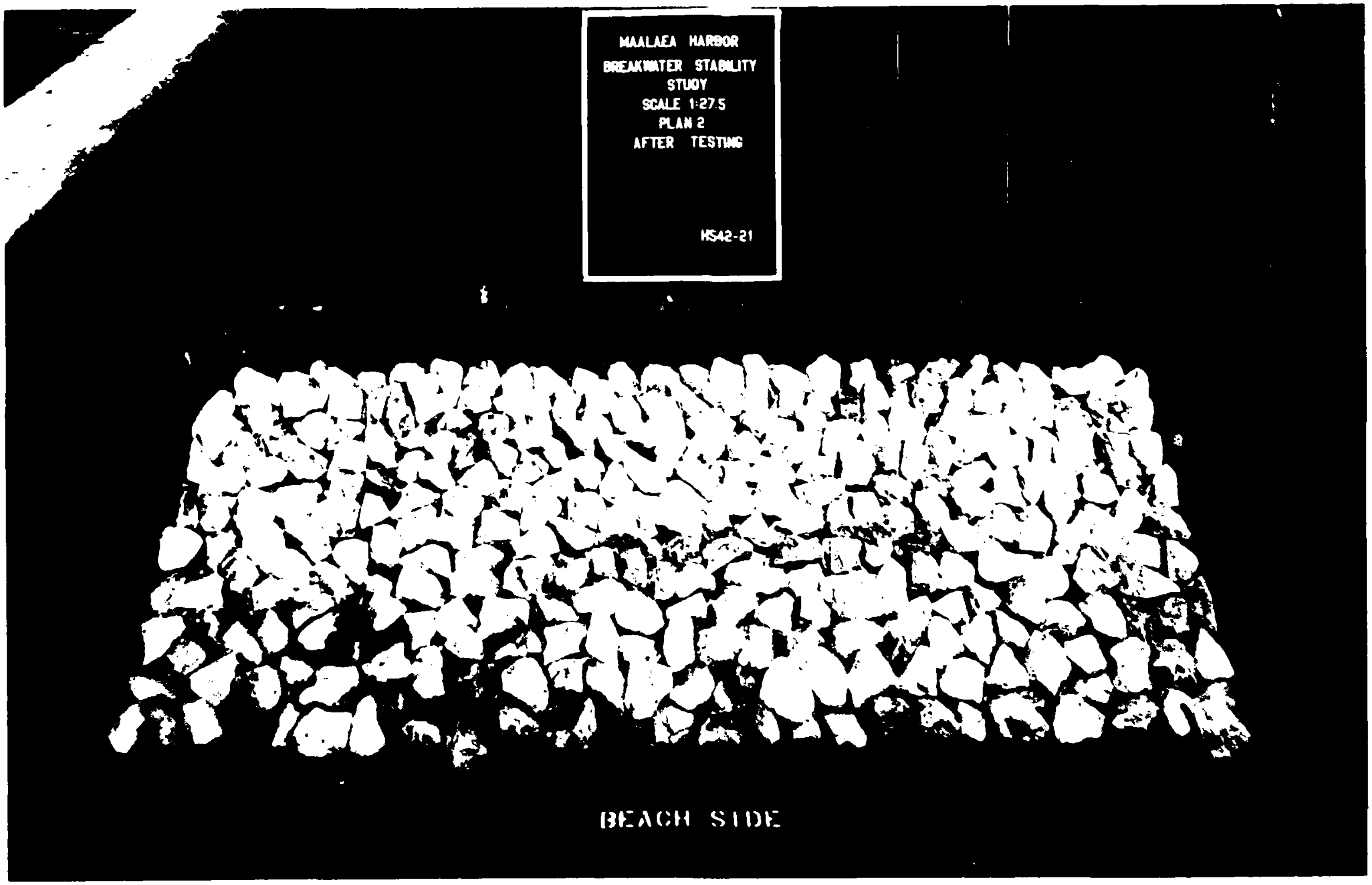


Photo 1[illegible]. Sea-side view of Plan 2 after compl[illegible]


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 2
AFTER TESTING
H542-16
BEACH SIDE


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 2
AFTER TESTING
H542-23


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 2
AFTER TESTING
H542-22
SEA SIDE



[illegible] attack of [illegible]-sec, [illegible]-ft breaking waves at an [illegible]

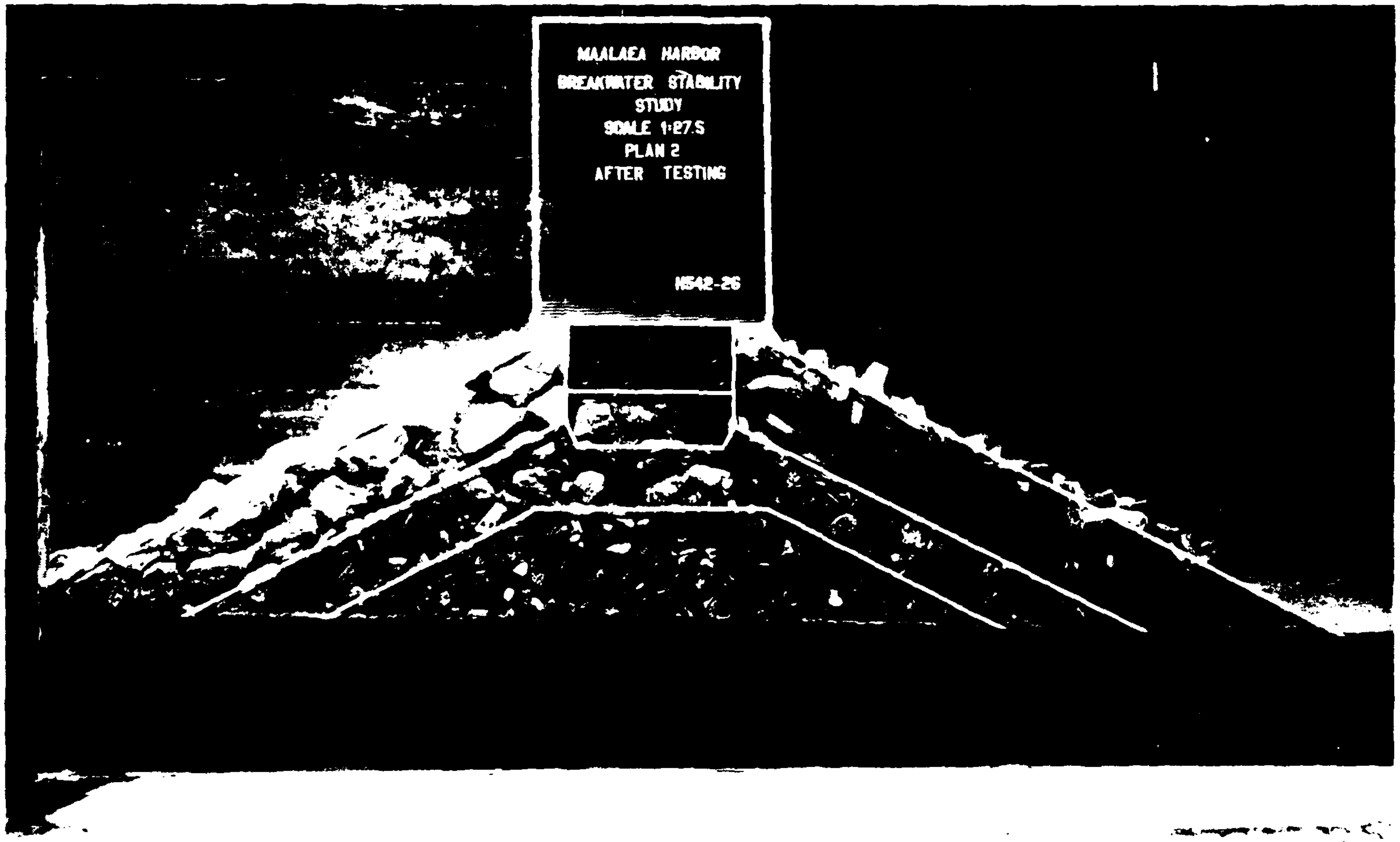


Photo [illegible]. Side view of Plan 2 after attack of [illegible], [illegible]
[illegible]


AFTER TESTING
SEA SIDE

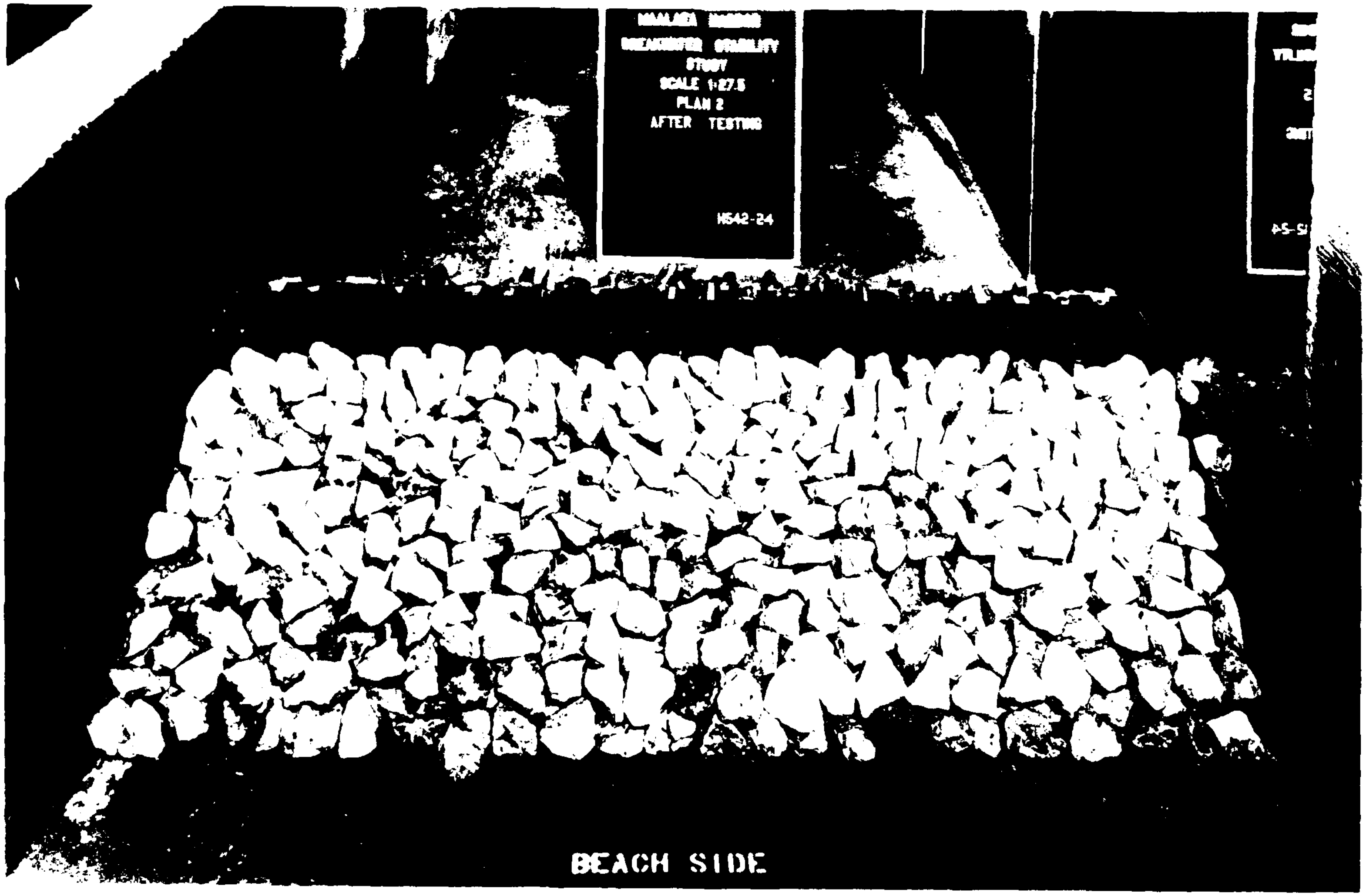
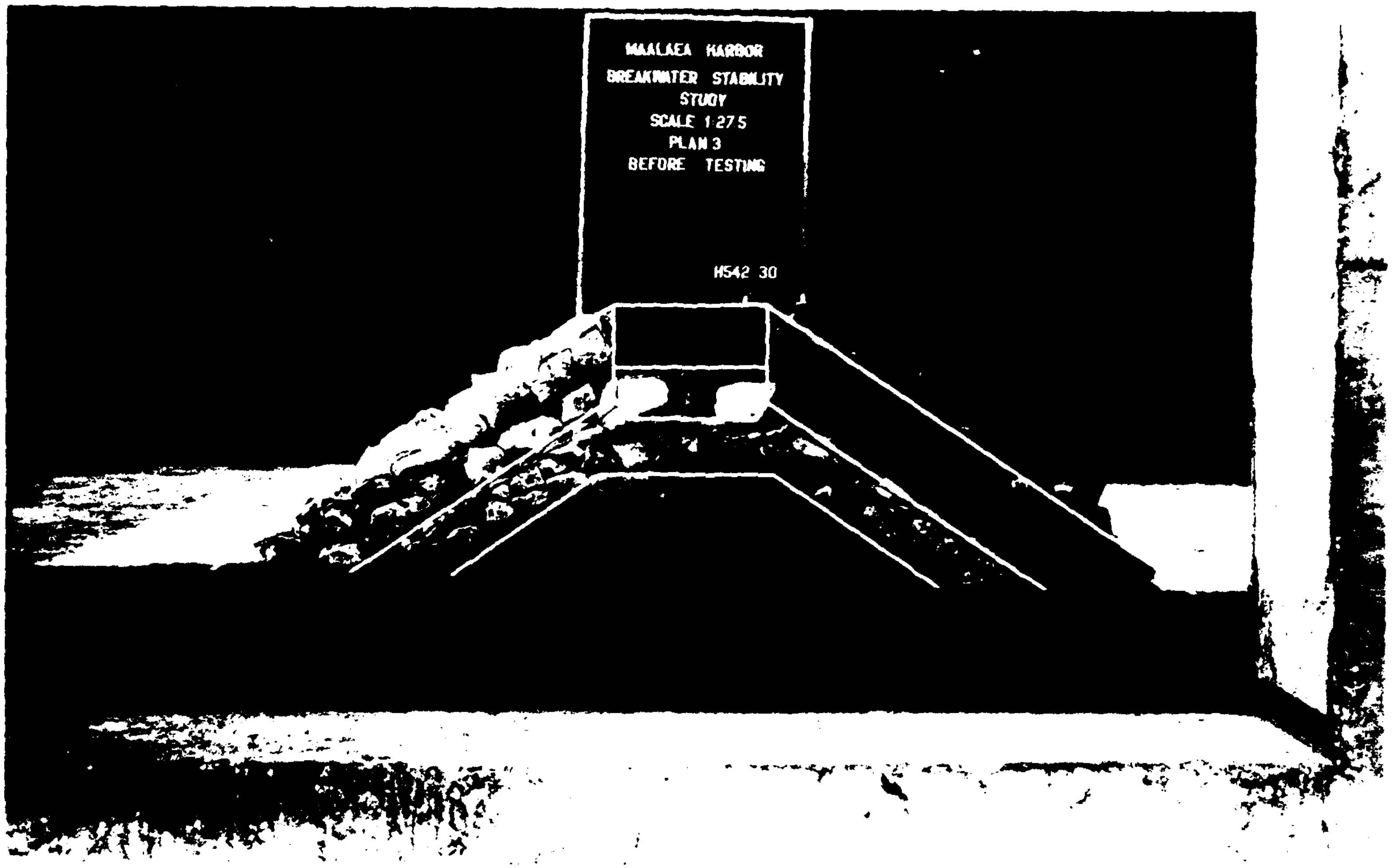
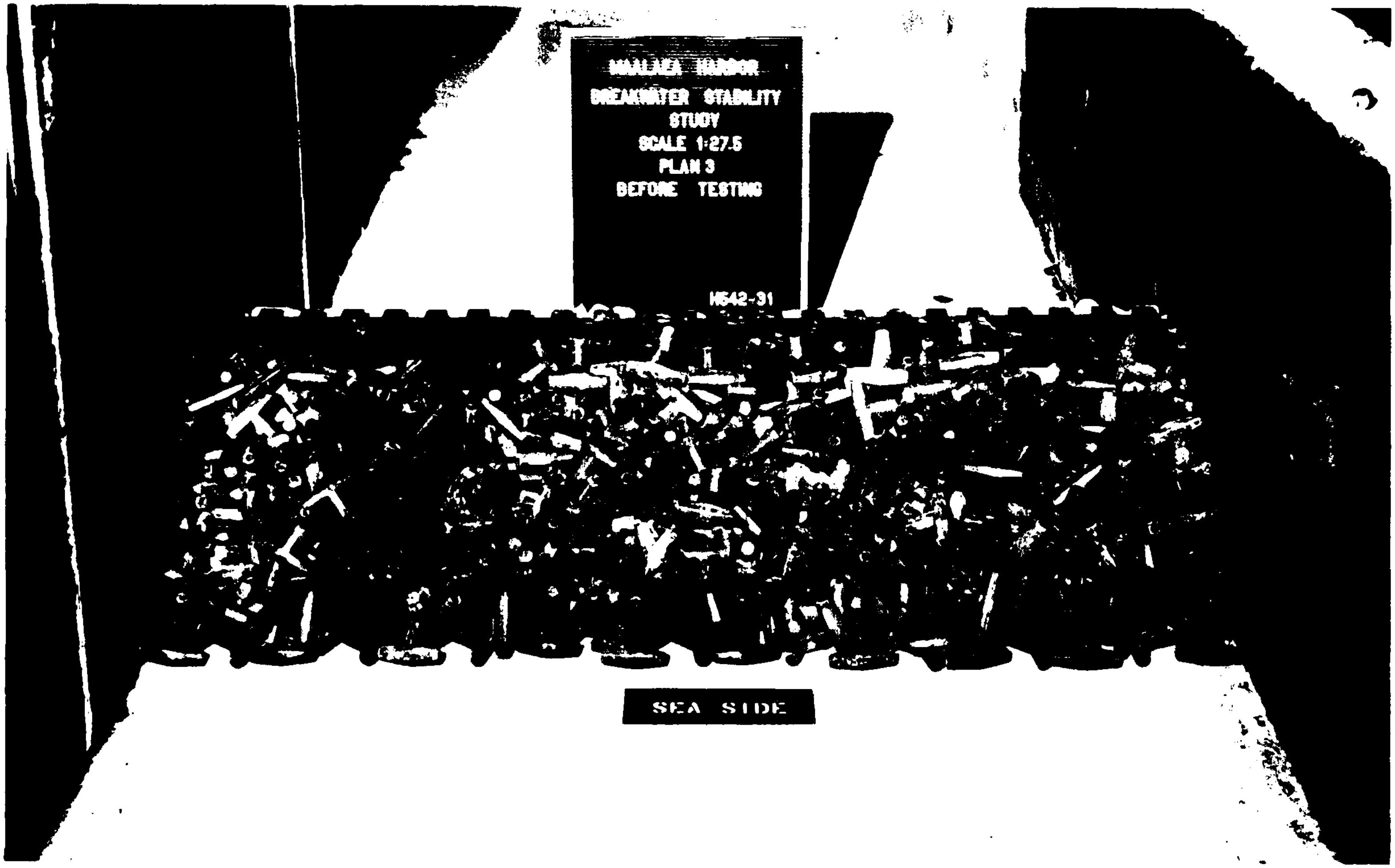
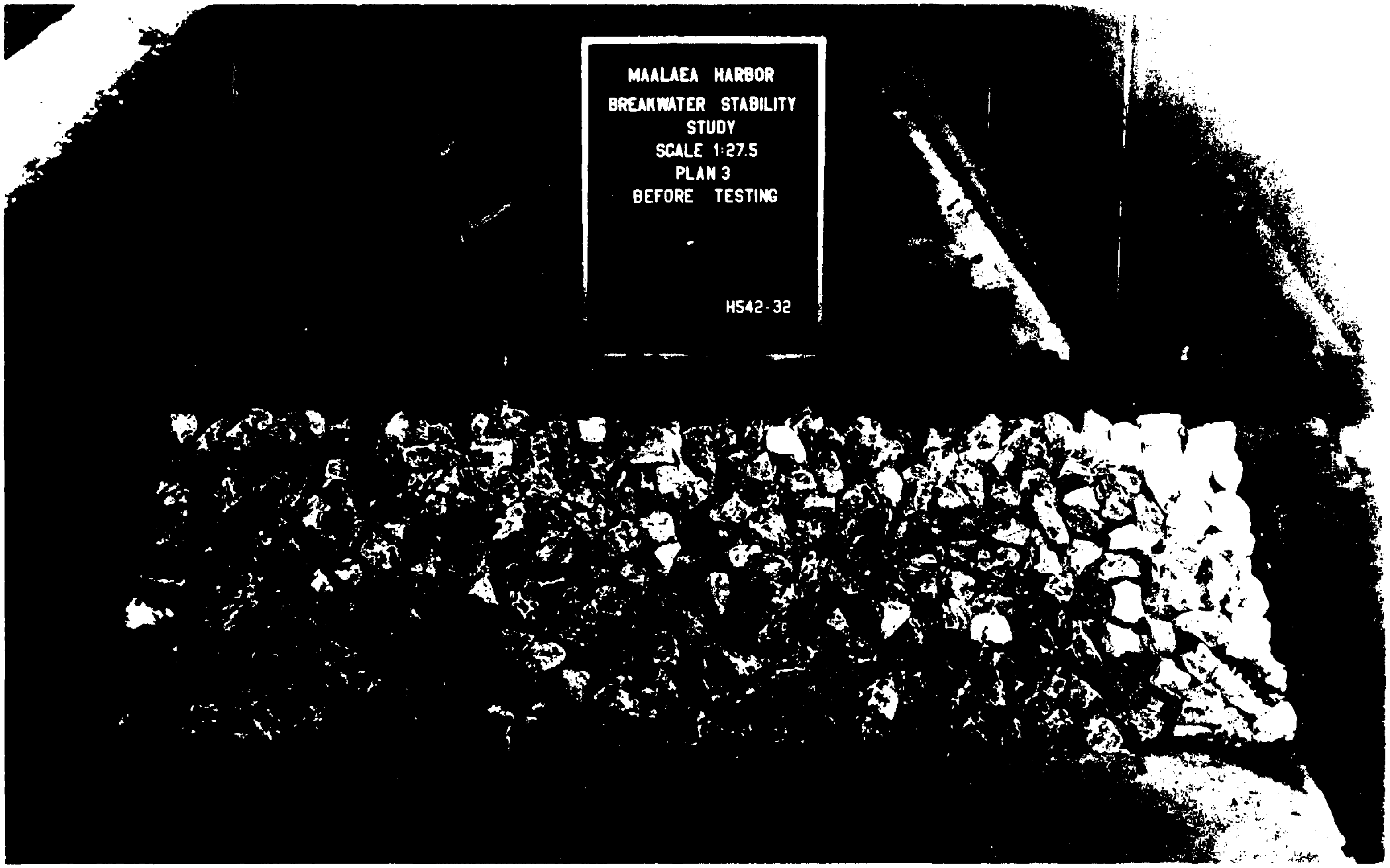
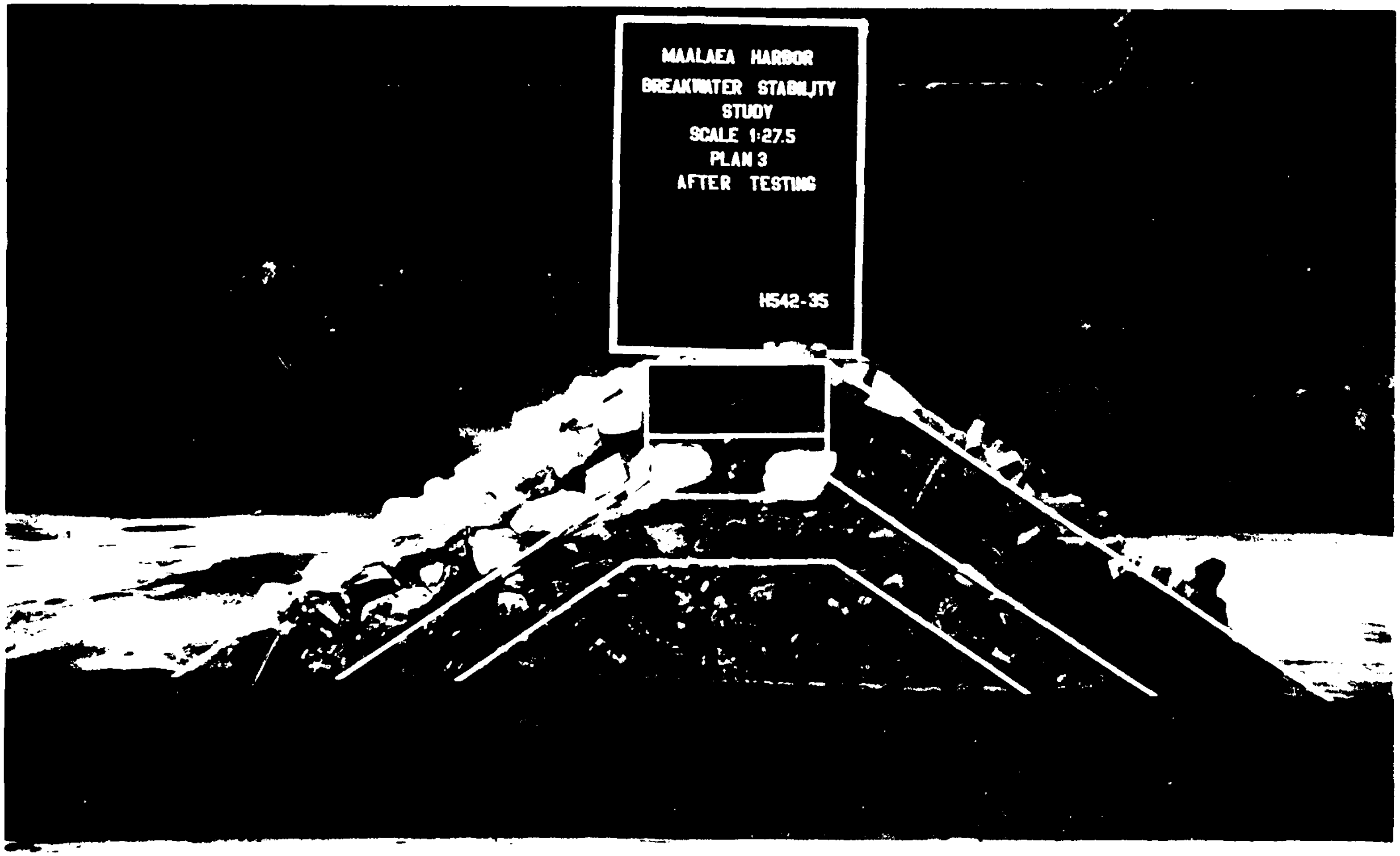


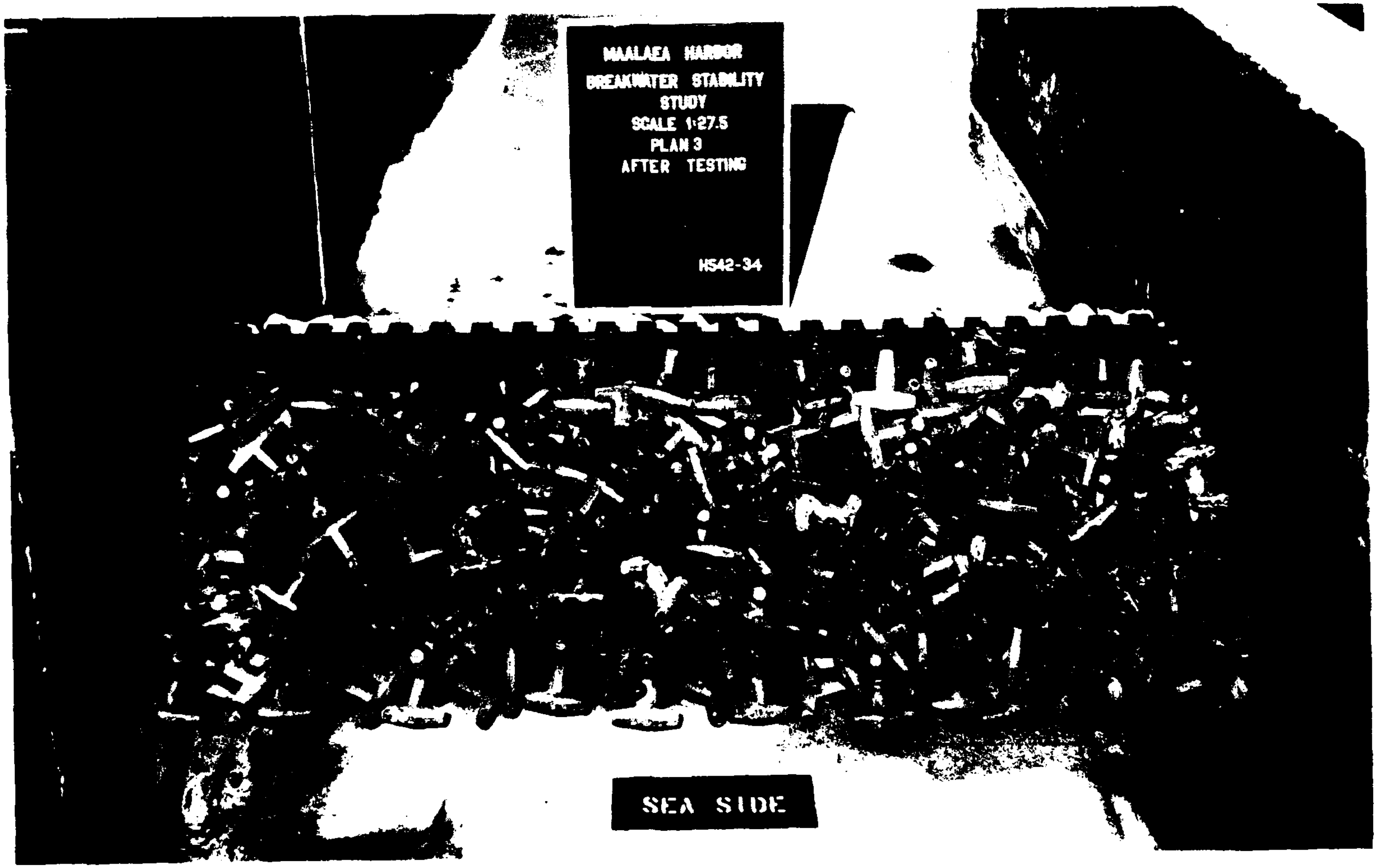
Photo [illegible]. Beach-side view of Plan 2 after attack of 16-sec, 16.[illegible]-ft breaking waves at an swl of +[illegible] ft mllw

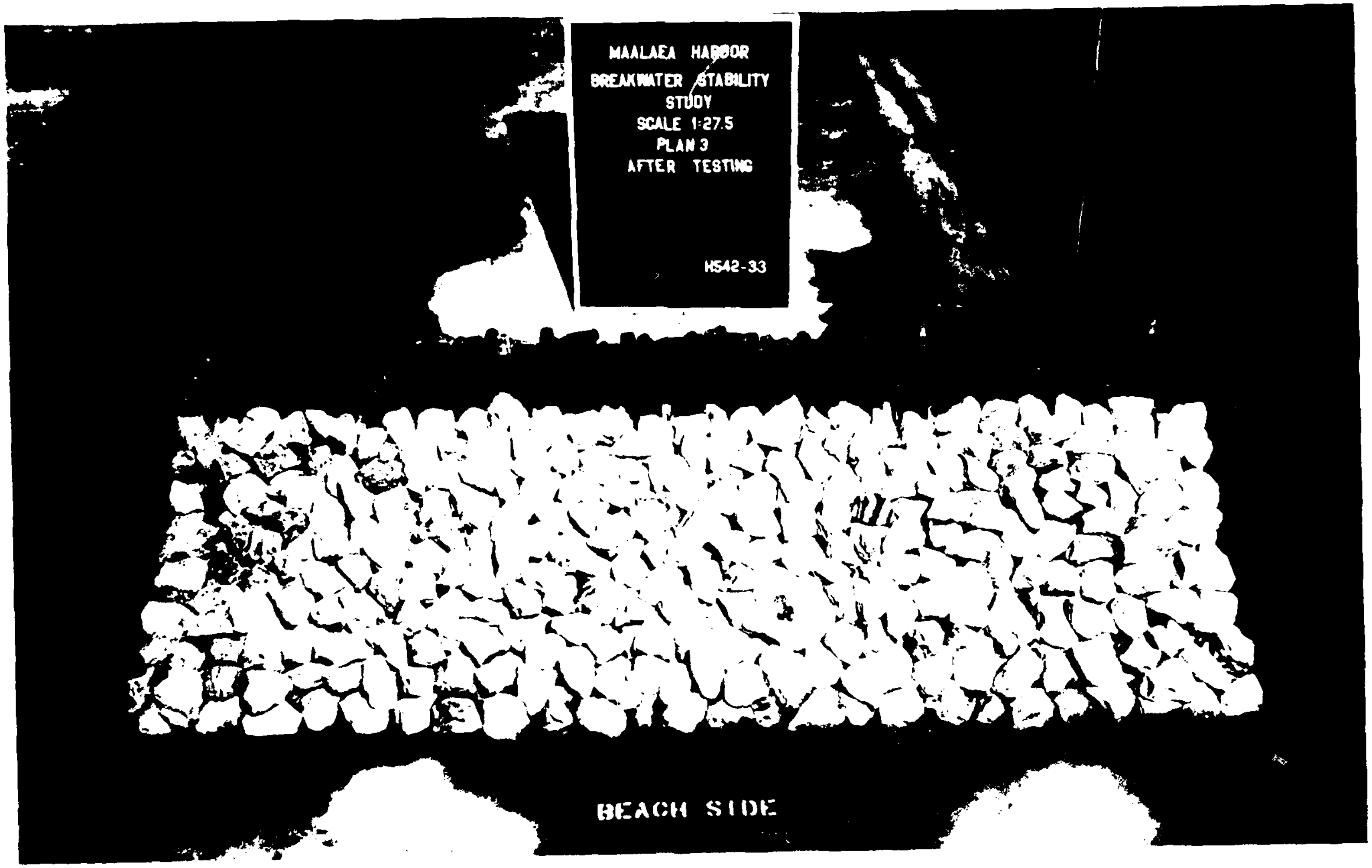

MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
BEFORE TESTING
H542 30


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
BEFORE TESTING
H542-31
SEA SIDE


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
BEFORE TESTING
H542-32


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
AFTER TESTING
H542-35


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
AFTER TESTING
H542-34
SEA SIDE


MAALAEA HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:27.5
PLAN 3
AFTER TESTING
H542-33
BEACH SIDE

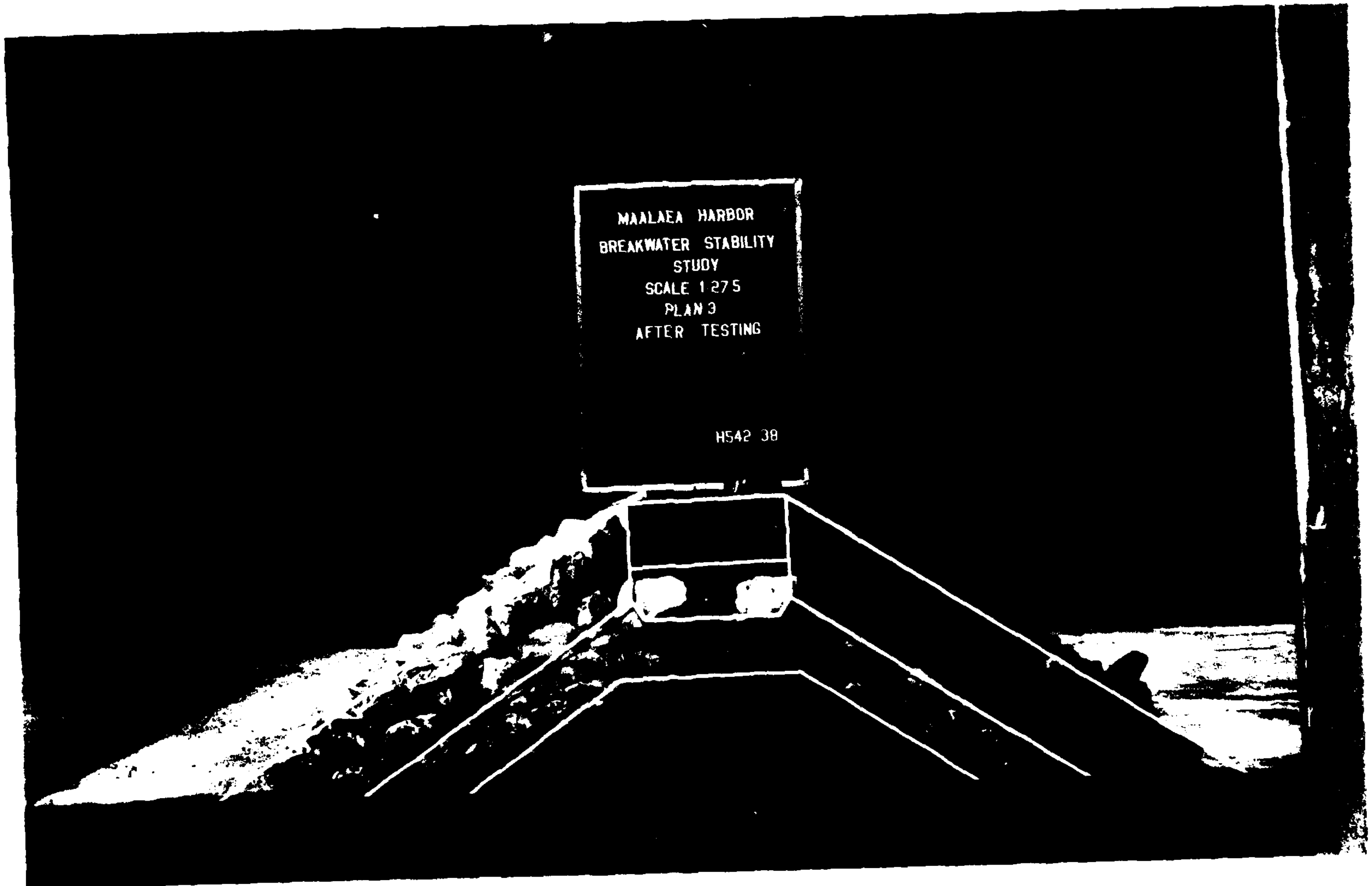


Photo [illegible]. Side view of Plan 3 after attack of [illegible]

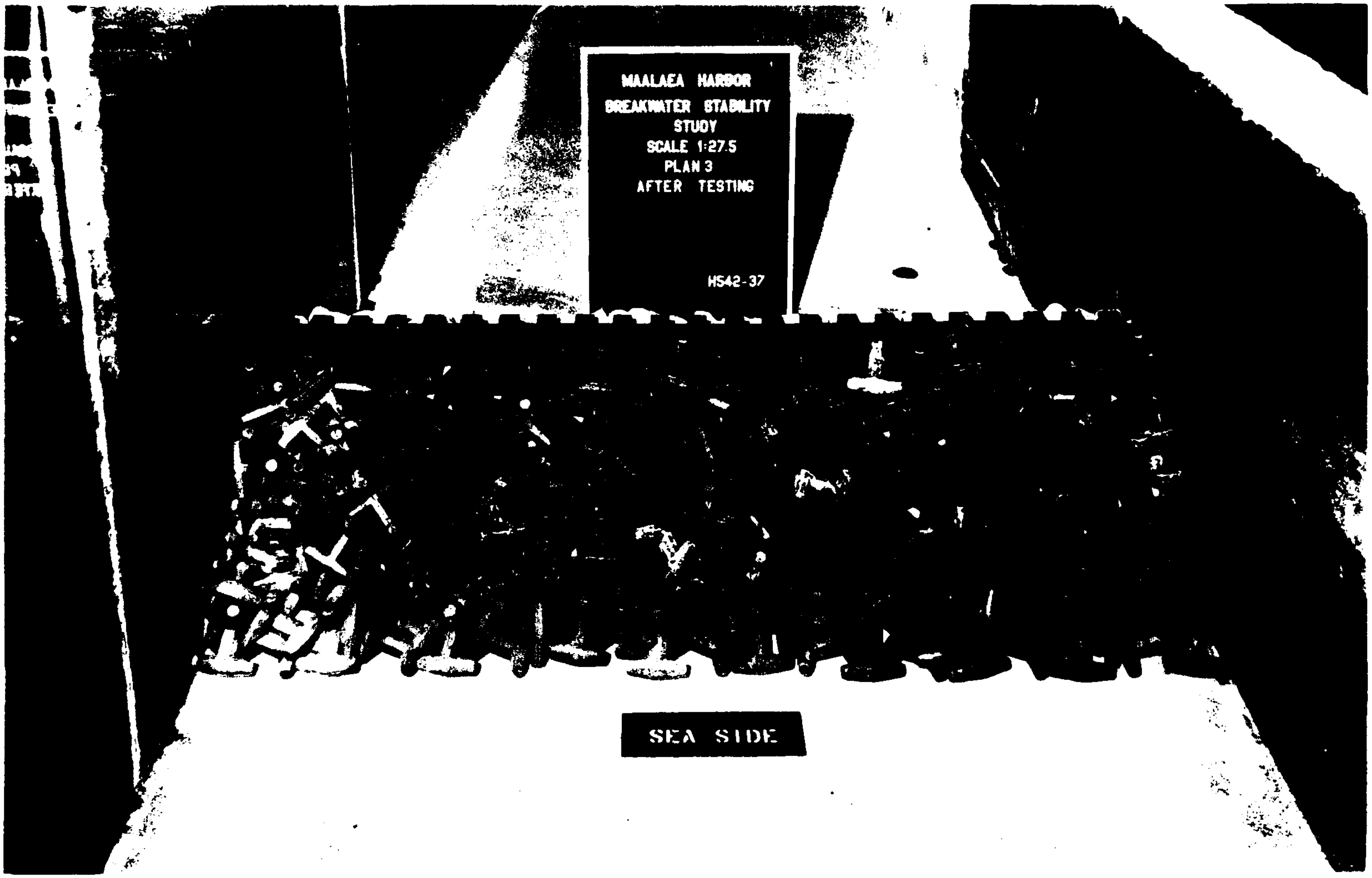


Photo 23. Sea-side view of Plan 3 after attack of [illegible]-sec, [illegible]-ft breaking waves at an [illegible]

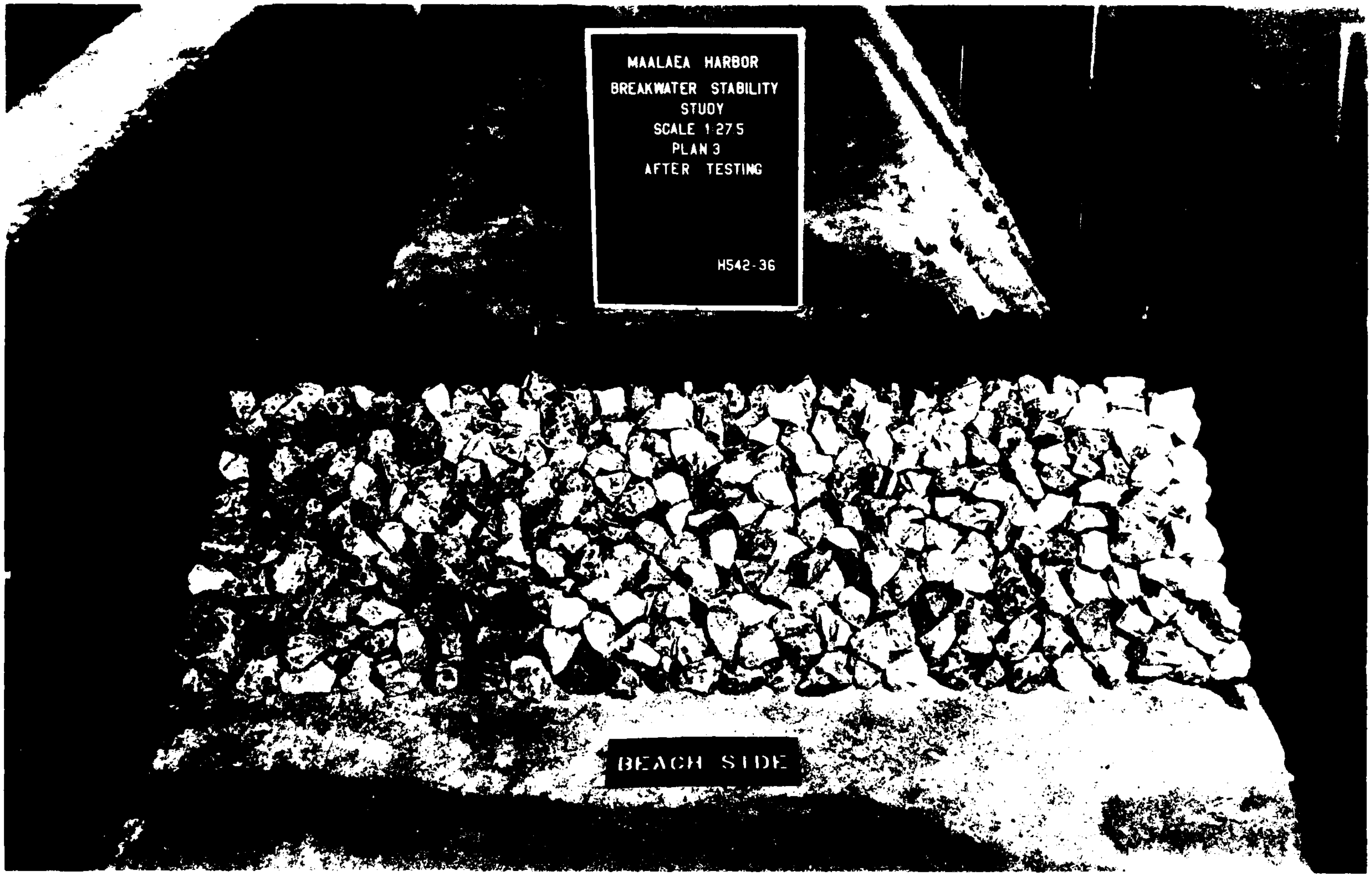


Photo [illegible]. Beach-side view of Plan 3 after attack [illegible]-sec, [illegible]-ft breaking waves at an swl [illegible] + [illegible] ft mllw

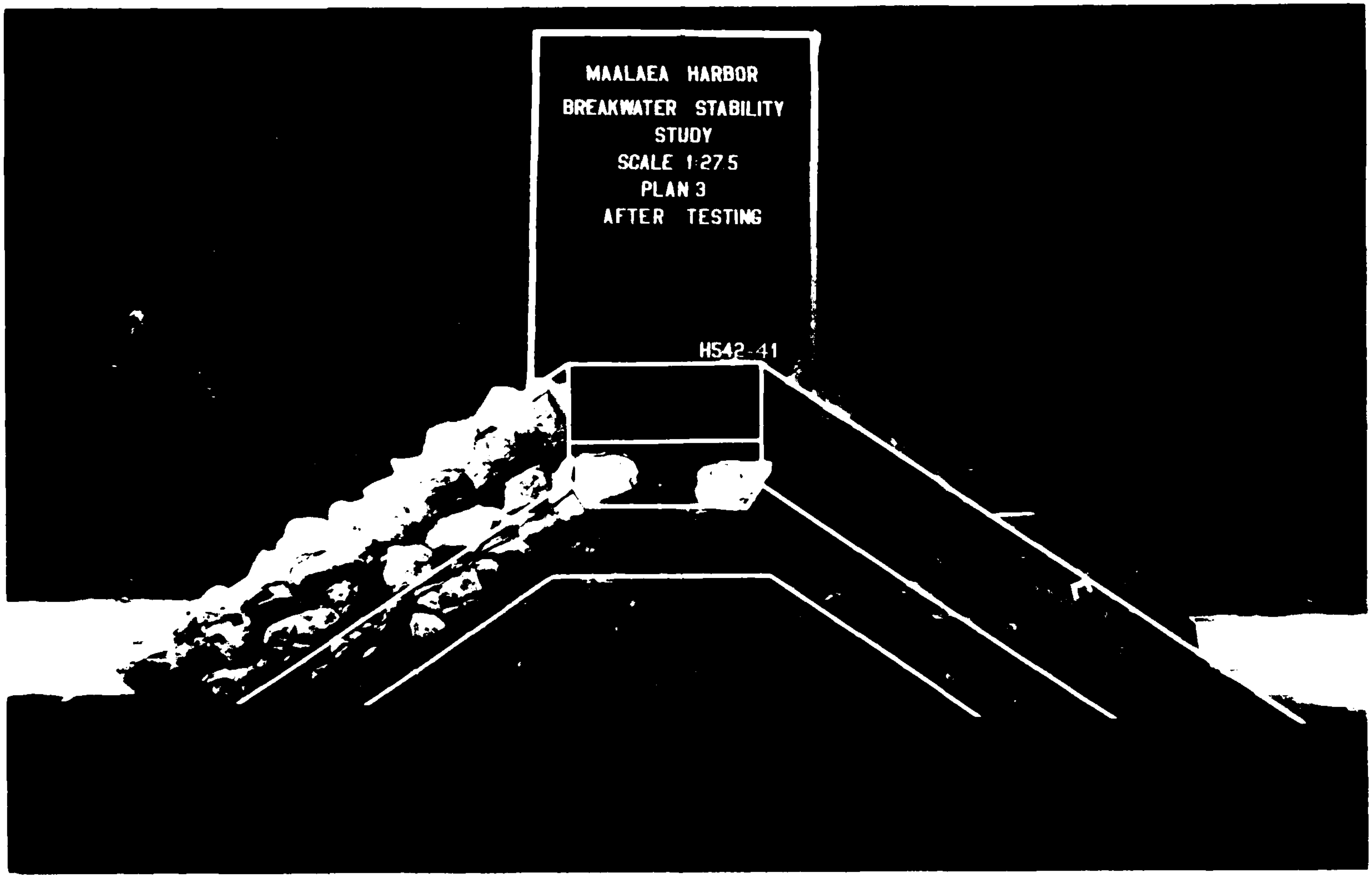


Photo 3[illegible]. Side view of Plan 3 after attack of 16-sec, 16.7-ft breaking waves at an swl of +[illegible] ft mllw

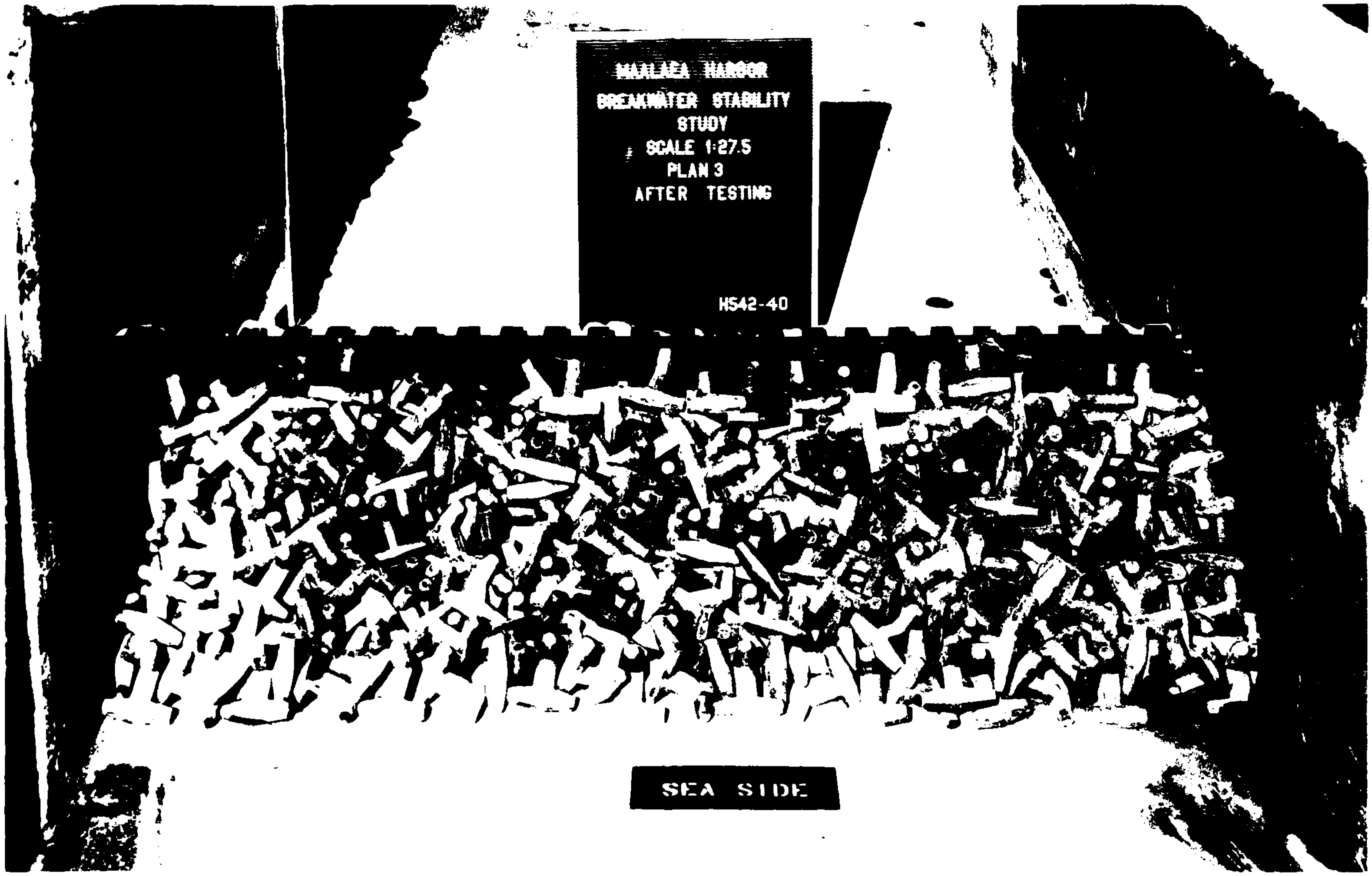


Photo [illegible]. Sea-side view of Plan 3 after attack of [illegible]-sec, 16.7-ft breaking waves at an swl of +[illegible] ft mllw

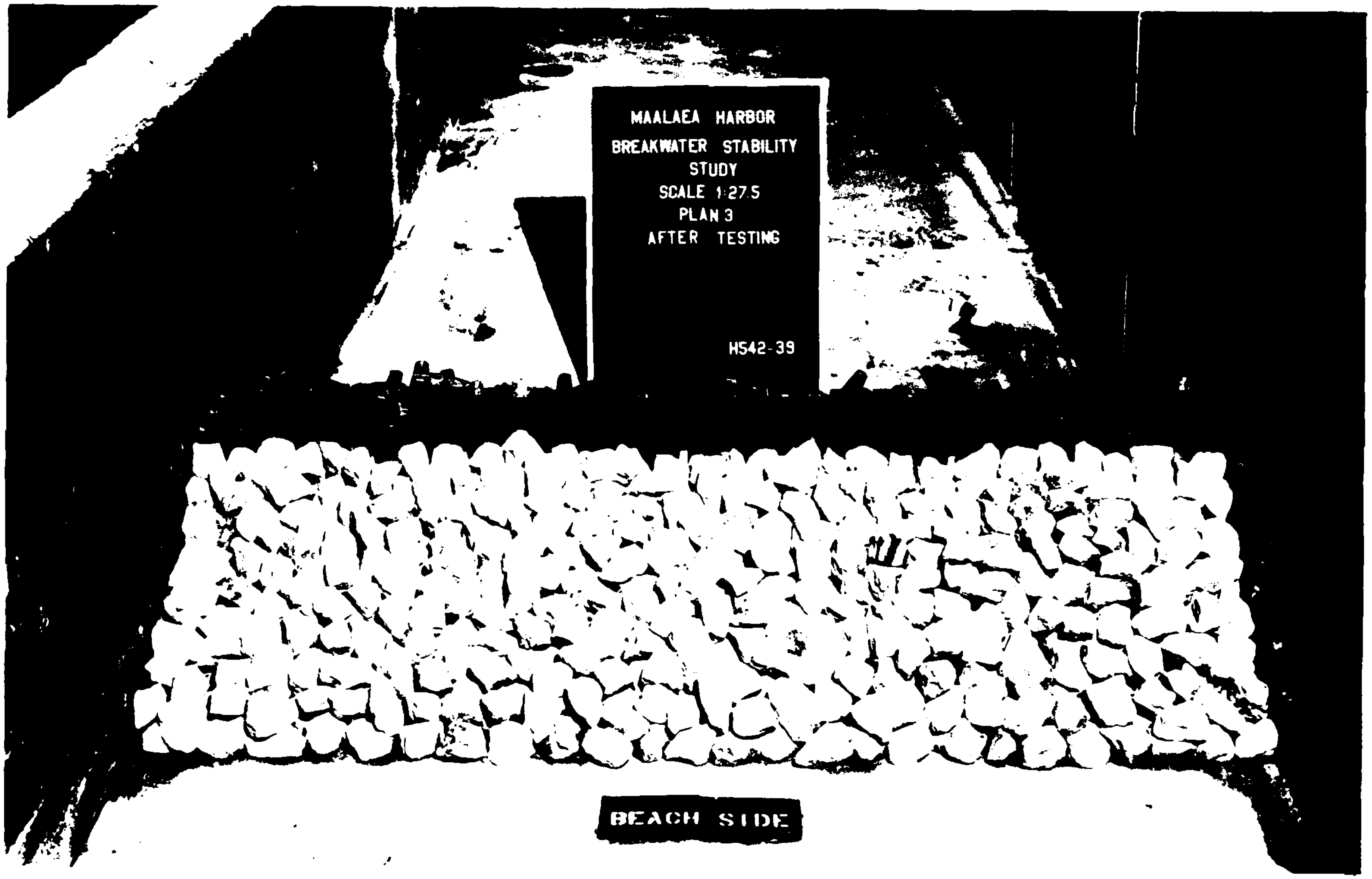


Photo 38. Beach-side view of Plan 3 after attack of 16-sec, 16.7-ft breaking waves at an swl of +3 ft mllw

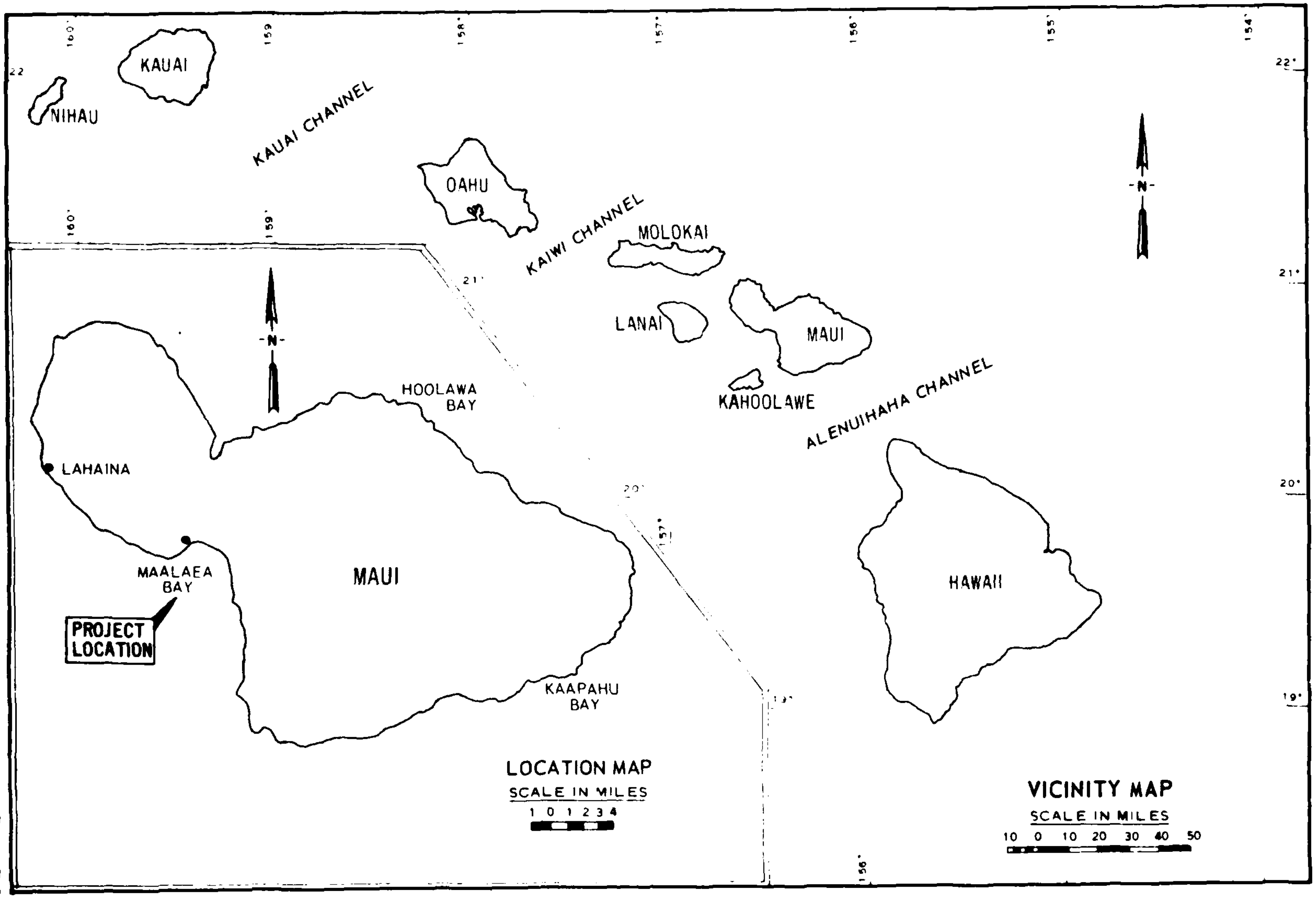
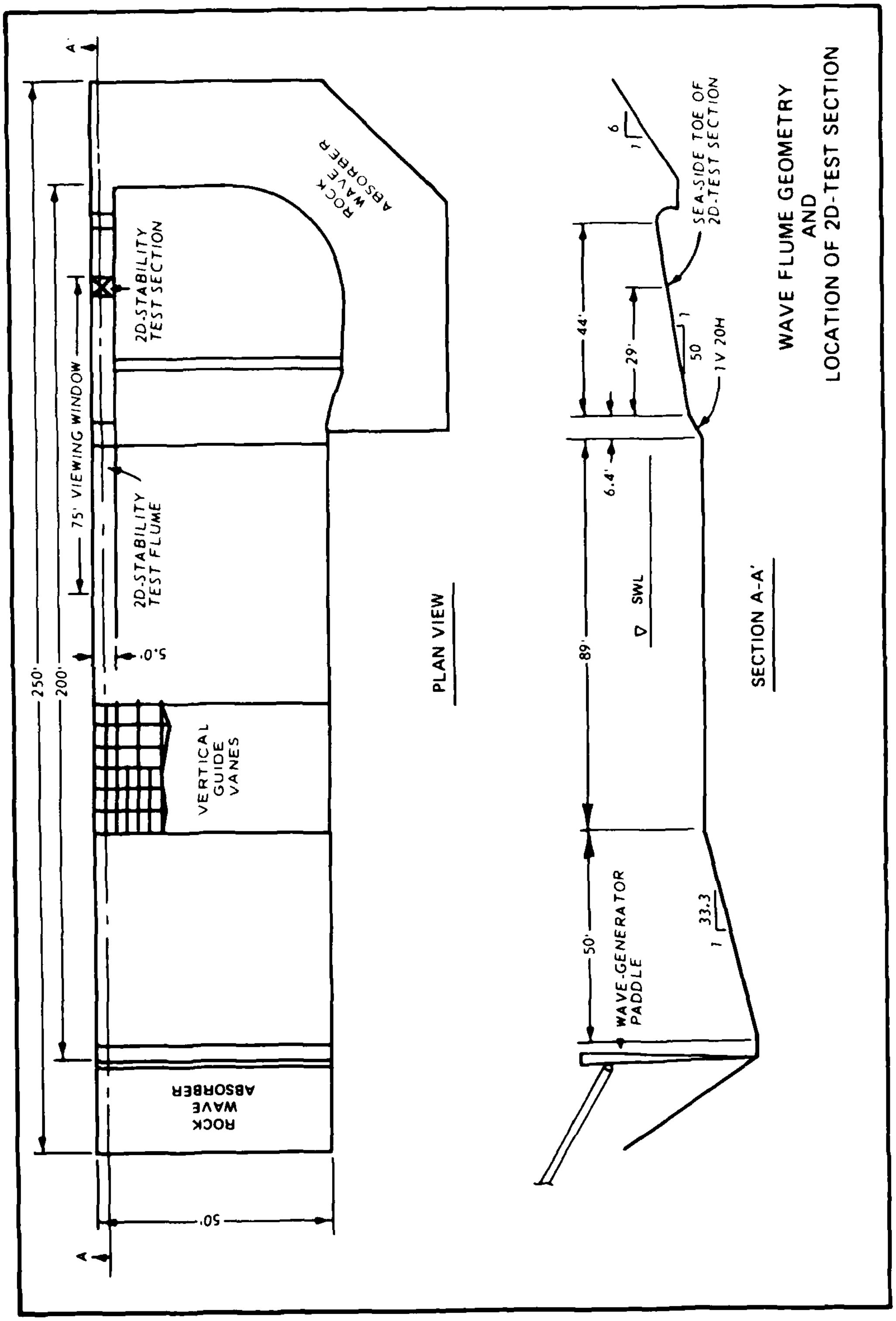
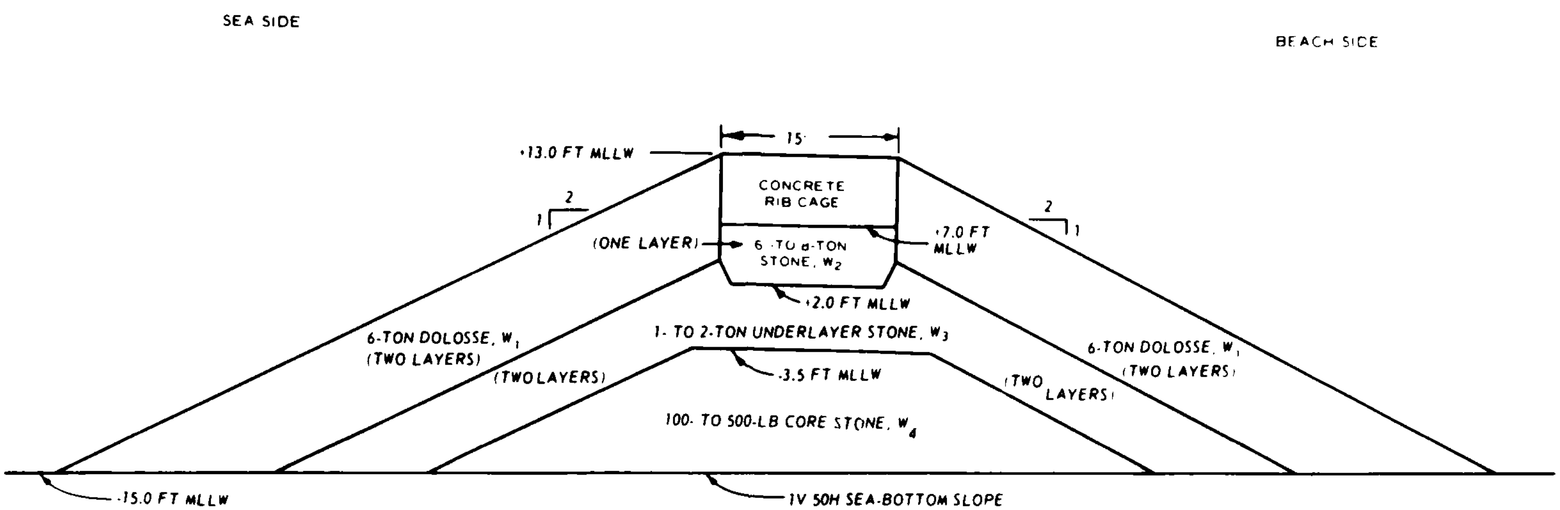

KAUAI
NIHAU
KAUAI CHANNEL
OAHU
KAIWI CHANNEL
MOLOKAI
LANAI
MAUI
KAHOOLAWE
ALENUIHAHA CHANNEL
HAWAII
HOOLAWA BAY
LAHAINA
MAALAEA BAY
PROJECT LOCATION
MAUI
KAAPAHU BAY
LOCATION MAP
SCALE IN MILES
1 0 1 2 3 4
VICINITY MAP
SCALE IN MILES
10 0 10 20 30 40 50
-N-

PLATE 1



WAVE FLUME GEOMETRY AND LOCATION OF 2D-TEST SECTION

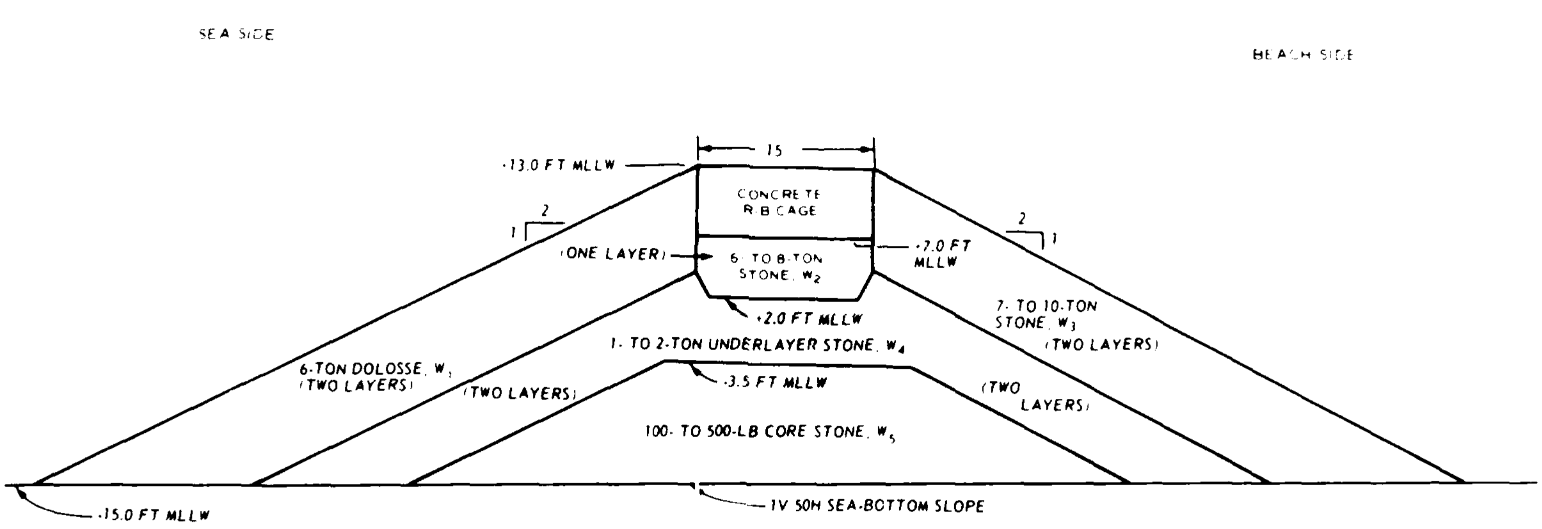
PLATE 2



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.59-LB DOLOSSE @ 141 PCF	W_1 = 6-TON DOLOSSE @ 147 PCF
W_2 = 0.365-LB TO 0.486-LB STONE @ 165 PCF	W_2 = 6- TO 8-TON STONE @ 156 PCF
W_3 = 0.061-LB TO 0.121-LB STONE @ 165 PCF	W_3 = 1- TO 2-TON STONE @ 156 PCF
W_4 = 0.003-LB TO 0.015-LB STONE @ 165 PCF	W_4 = 100- TO 500-LB STONE @ 156 PCF

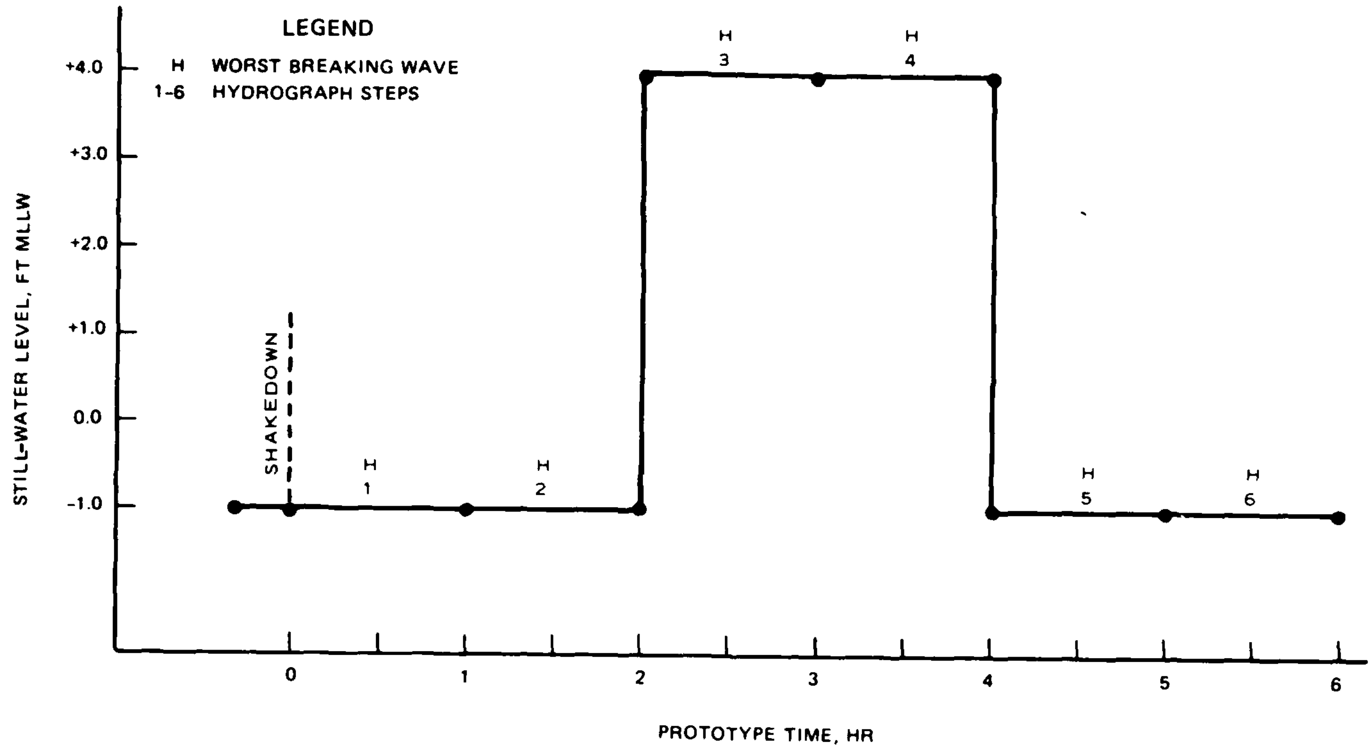
PLAN 1
TYPICAL BREAKWATER CROSS SECTION
MODEL SCALE 1:27.5



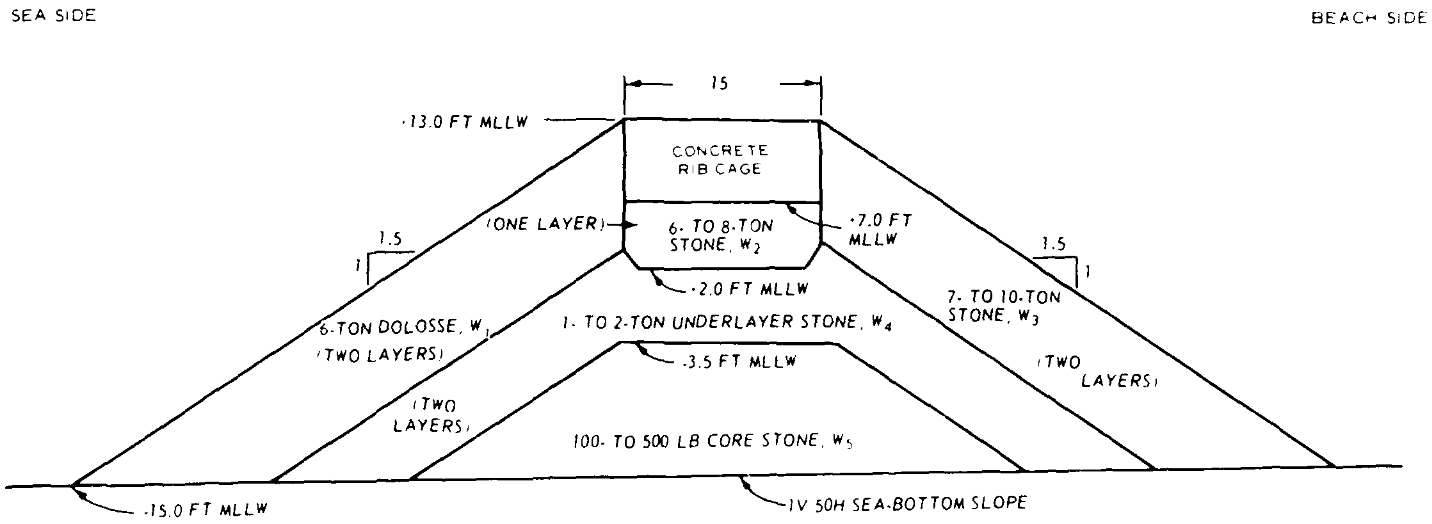
MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.59-LB DOLOSSE @ 141 PCF	W_1 = 6-TON DOLOSSE @ 147 PCF
W_2 = 0.365-LB TO 0.486-LB STONE @ 165 PCF	W_2 = 6- TO 8-TON STONE @ 156 PCF
W_3 = 0.426-LB TO 0.608-LB STONE @ 165 PCF	W_3 = 7- TO 10-TON STONE @ 156 PCF
W_4 = 0.061-LB TO 0.121-LB STONE @ 165 PCF	W_4 = 1- TO 2-TON STONE @ 156 PCF
W_5 = 0.003-LB TO 0.015-LB STONE @ 165 PCF	W_5 = 100- TO 500-LB STONE @ 156 PCF

PLAN 2
TYPICAL BREAKWATER CROSS SECTION
MODEL SCALE 1:27.5



STORM SURGE HYDROGRAPH



MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
W_1 = 0.59-LB DOLOSSE @ 141 PCF	W_1 6-TON DOLOSSE @ 147 PCF
W_2 = 0.365-LB TO 0.486-LB STONE @ 165 PCF	W_2 6- TO 8-TON STONE @ 156 PCF
W_3 = 0.426-LB TO 0.608 LB STONE @ 165 PCF	W_3 = 7- TO 10-TON STONE @ 156 PCF
W_4 = 0.061-LB TO 0.121-LB STONE @ 165 PCF	W_4 1- TO 2-TON STONE @ 156 PCF
W_5 0.003 LB TO 0.015 LB STONE @ 165 PCF	W_5 100- TO 500-LB STONE @ 156 PCF

PLAN 3

TYPICAL BREAKWATER CROSS SECTION

MODEL SCALE 1:27.5

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Carver, Robert D

Stability of rubble-mound breakwater Maalaea Harbor, Maui, Hawaii; hydraulic model investigation / by Robert D. Carver, Dennis G. Markle. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1981.

12, [32] p. [6] leaves of plates : ill. ; 27 cm. (Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; HL-81-1)

Prepared for U. S. Army Engineer Division, Pacific Ocean, Fort Shafter, Hawaii.

1. Breakers (Water waves). 2. Breakwaters. 3. Hydraulic models. 4. Maalaea Harbor, Hawaii. 5. Rubble-mound breakwaters. I. Markle, Dennis G., joint author. II. United States. Army. Corps of Engineers. Pacific Ocean Division. III. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; HL-81-1.

TA7.W34m no.HL-81-1

DATE
FILMED